DRAFT
CULVERTS AND BRIDGES

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1.0 INTRODUCTION AND OVERVIEW

The function of a culvert is to convey surface water across a highway, railroad, or other embankment. In addition to the hydraulic function, the culvert must carry construction, highway, railroad, or other traffic and earth loads. Therefore, culvert design involves both hydraulic and structural design considerations. The hydraulic aspects of culvert design are set forth in this chapter.

Culverts are available in a variety of sizes, shapes, and materials. These factors, along with several others, affect their capacity and overall performance. Sizes and shapes may vary from small circular pipes to extremely large arch sections that are sometimes used in place of bridges.

The most commonly used culvert shape is circular, but arches, boxes, and elliptical shapes are used, as well. Pipe arch, elliptical, and rectangular shapes are generally used in lieu of circular pipe where there is limited cover. Arch culverts have application in locations where less obstruction to a waterway is a desirable feature, and where foundations are adequate for structural support. Box culverts can be designed to pass large flows and to fit nearly any site condition. A box or rectangular culvert lends itself more readily than other shapes to low allowable headwater situations since the height may be decreased and the span increased to satisfy the location requirements.

The material selected for a culvert is dependent upon various factors, such as durability, structural strength, roughness, bedding condition, abrasion and corrosion resistance, and water tightness. The more common culvert materials used are concrete and steel (smooth and corrugated).

Another factor that significantly affects the performance of a culvert is its inlet configuration. The culvert inlet may consist of a culvert barrel projecting from the roadway fill or mitered to the embankment slope. Other inlets have headwalls, wingwalls, and apron slabs or standard end sections of concrete or metal.

A careful approach to culvert design is essential, both in new land development and retrofit situations, because culverts often significantly influence upstream and downstream flood risks, floodplain management, and public safety (Photograph 1). Culverts can be designed to provide beneficial upstream conditions (Photograph 2) and to avoid negative visual impact (Photograph 3).

Photograph 1—Public safety considerations for long culverts should be accounted for with culvert designs such as with this collapsible trash rack at a park-like location.

Photograph 2—Culverts can be designed to provide compatible upstream conditions for desirable wetland growth.

Photograph 3—Culverts can be integrated into the urban landscape without negative visual impact.

The information and references necessary to design culverts according to the procedure given in this chapter can be found in *Hydraulic Design of Highway Culverts*, Hydraulic Design Series No. 5 (FHWA 1985). Some of the charts and nomographs from that publication covering the more common requirements are given in this chapter. Nomographs and charts covering the range of applications commonly encountered in urban drainage are contained in Section 11.0. For special cases and larger sizes, the FHWA publication should be used.

1.1 Required Design Information

The hydraulic design of a culvert essentially consists of an analysis of the required performance of the culvert to convey flow from one side of the roadway (or other kind of embankment, such as a railroad) to the other. The designer must select a design flood frequency, estimate the design discharge for that frequency, and set an allowable headwater elevation based on the selected design flood and headwater considerations. These criteria are typically dictated by local requirements although state and federal standards will apply to relevant highway projects. The culvert size and type can be selected after the design discharge, controlling design headwater, slope, tailwater, and allowable outlet velocity have been determined.

The design of a culvert includes a determination of the following:

- Impacts of various culvert sizes and dimensions on upstream and downstream flood risks, including the implications of embankment overtopping.
- How will the proposed culvert/embankment fit into the relevant major drainageway master plan, and are there multipurpose objectives that should be satisfied?
- Alignment, grade, and length of culvert.
- Size, type, end treatment, headwater, and outlet velocity.
- Amount and type of cover.
- Public safety issues, including the key question of whether or not to include a safety/debris rack (Photograph 4).
- Pipe material.
- Type of coating (if required).
- Need for protective measures against abrasion and corrosion.
- Need for specially designed inlets or outlets.
- Structural and geotechnical considerations, which are beyond the scope of this chapter.

Photograph 4—Public safety features such as the rack at the entrance to an irrigation ditch and the railing on the wingwalls must be considered.

1.1.1 Discharge

The discharge used in culvert design is usually estimated on the basis of a preselected storm recurrence interval, and the culvert is designed to operate within acceptable limits of risk at that flow rate. The design recurrence interval should be based on the criteria set forth in this *Manual*.

1.1.2 Headwater

Culverts generally constrict the natural stream flow, which causes a rise in the upstream water surface. The elevation of this water surface is termed *headwater elevation*, and the total flow depth in the stream measured from the culvert inlet invert is termed *headwater depth*.

In selecting the design headwater elevation, the designer should consider the following:

- Anticipated upstream and downstream flood risks, for a range of return frequency events.
- Damage to the culvert and the roadway.
- Traffic interruption.
- Hazard to human life and safety.
- Headwater/Culvert Depth (HW/D) ratio.
- Low point in the roadway grade line.
- Roadway elevation above the structure.
- Elevation at which water will flow to the next cross drainage.

- Relationship to stability of embankment that culvert passes through.

The headwater elevation for the design discharge should be consistent with the freeboard and overtopping criteria in the __________ chapter of this *Manual*. The designer should verify that the watershed divides are higher than the design headwater elevations. In flat terrain, drainage divides are often undefined or nonexistent and culverts should be located and designed for the least disruption of the existing flow distribution.

1.1.3 Tailwater

Tailwater is the flow depth in the downstream channel measured from the invert at the culvert outlet. It can be an important factor in culvert hydraulic design because a submerged outlet may cause the culvert to flow full rather than partially full.

A field inspection of the downstream channel should be made to determine whether there are obstructions that will influence the tailwater depth. Tailwater depth may be controlled by the stage in a contributing stream, headwater from structures downstream of the culvert, reservoir water surface elevations, or other downstream features.

1.1.4 Outlet Velocity

The outlet velocity of a highway culvert is the velocity measured at the downstream end of the culvert, and it is usually higher than the maximum natural stream velocity. This higher velocity can cause streambed scour and bank erosion for a limited distance downstream from the culvert outlet. Permissible velocities at the outlet will depend upon streambed type, and the kind of energy dissipation (outlet protection) that is provided.

If the outlet velocity of a culvert is too high, it may be reduced by changing the barrel roughness. If this does not give a satisfactory reduction, it may be necessary to use some type of outlet protection or energy dissipation device. Most culverts require adequate outlet protection, and this is a frequently overlooked issue during design.

Variations in shape and size of a culvert seldom have a significant effect on the outlet velocity. Slope and roughness of the culvert barrel are the principal factors affecting the outlet velocity.

2.0 CULVERT HYDRAULICS

2.1 Key Hydraulic Principles

For purposes of the following review, it is assumed that the reader has a basic working knowledge of hydraulics and is familiar with the Manning's (Equation 1), continuity (Equation 2), and energy (Equation 3) equations:

$$Q = \frac{1.49}{n} A R^{2/3} S^{1/2}$$

(Equation 1)

$$Q = v_1 A_1 = v_2 A_2$$

(Equation 2)

$$\frac{v^2}{2g} + \frac{p}{\gamma} + z + \text{losses} = \text{constant}$$

(Equation 3)

2.1.1 Energy and Hydraulic Grade Lines

Figures 1 and 2 illustrate the energy grade line (EGL) and hydraulic grade line (HGL) and related terms.

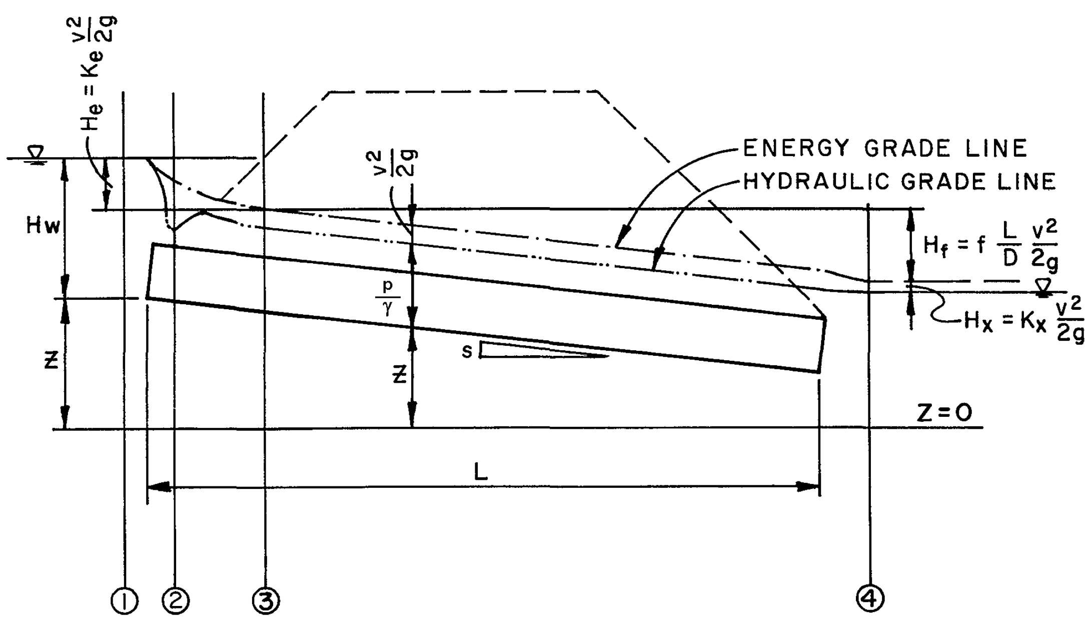


Figure 1—Definition of Terms for Closed Conduit Flow

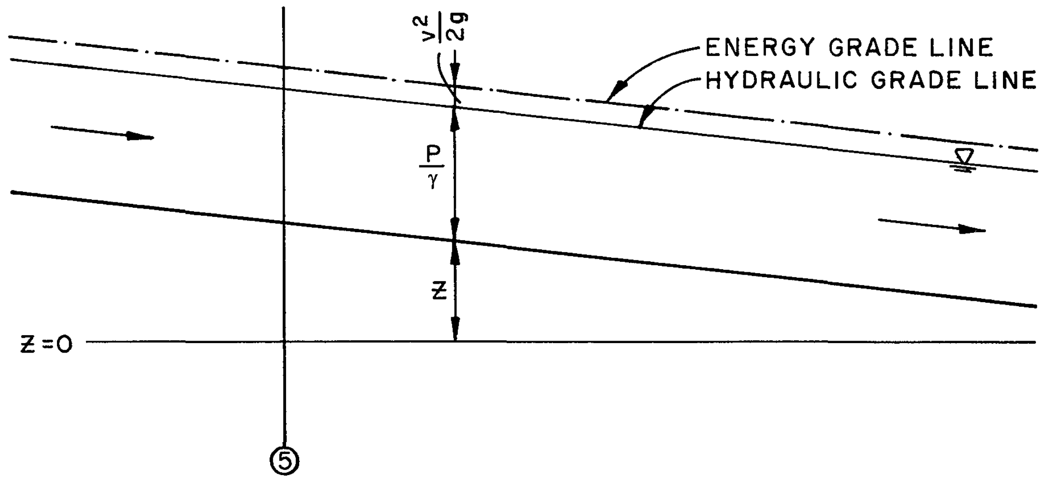


Figure 2—Definition of Terms for Open Channel Flow

The energy grade line, also known as the line of total head, is the sum of velocity head $v^2/2g$, the depth of flow or pressure head p/γ, and elevation above an arbitrary datum represented by the distance z. The energy grade line slopes downward in the direction of flow by an amount equal to the energy gradient H_L/L, where H_L equals the total energy loss over the distance L.

The hydraulic grade line, also known as the line of piezometric head, is the sum of the elevation z and the depth of flow or pressure head p/γ.

For open channel flow, the term p/γ is equivalent to the depth of flow and the hydraulic grade line is the same as the water surface. For pressure flow in conduits, p/γ is the pressure head and the hydraulic grade line falls above the top of the conduit as long as the pressure relative to atmospheric pressure is positive.

Approaching the entrance to a culvert as at Point 1 of Figure 1, the flow is essentially uniform and the hydraulic grade line and energy grade lines are almost the same. As water enters the culvert at the inlet, the flow is first contracted and then expanded by the inlet geometry causing a loss of energy at Point 2. As normal turbulent velocity distribution is reestablished downstream of the entrance at Point 3, a loss of energy is incurred through friction or form resistance. In short culverts, the entrance losses are likely to be high relative to the friction loss. At the exit, Point 4, an additional loss is incurred through turbulence as the flow expands and is retarded by the water in the downstream channel. At Point 5 of Figure 2 open channel flow is established and the hydraulic grade line is the same as the water surface.

There are two major types of flow conditions in culverts: (1) *inlet control* and (2) *outlet control*. For each type of control, a different combination of factors is used to determine the hydraulic capacity of a culvert. The determination of actual flow conditions can be difficult; therefore, the designer must check for both types of control and design for the most adverse condition.

2.1.2 Inlet Control

A culvert operates with inlet control when the flow capacity is controlled at the entrance by these factors:

- Depth of headwater
- Cross-sectional area
- Inlet edge configuration
- Barrel shape

When a culvert operates under inlet control, headwater depth and the inlet edge configuration determine the culvert capacity with the culvert barrel usually flowing only partially full.

Inlet control for culverts may occur in two ways. The least common occurs when the headwater depth is not sufficient to submerge the top of the culvert and the culvert invert slope is supercritical as shown in Figure 3.

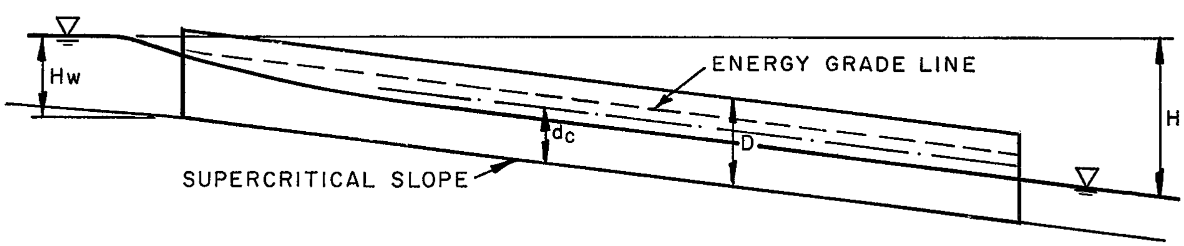


Figure 3—Inlet Control—Unsubmerged Inlet

The most common occurrence of inlet control is when the headwater submerges the top of the culvert (Figure 4), and the pipe does not flow full. A culvert flowing under inlet control is defined as a hydraulically short culvert.

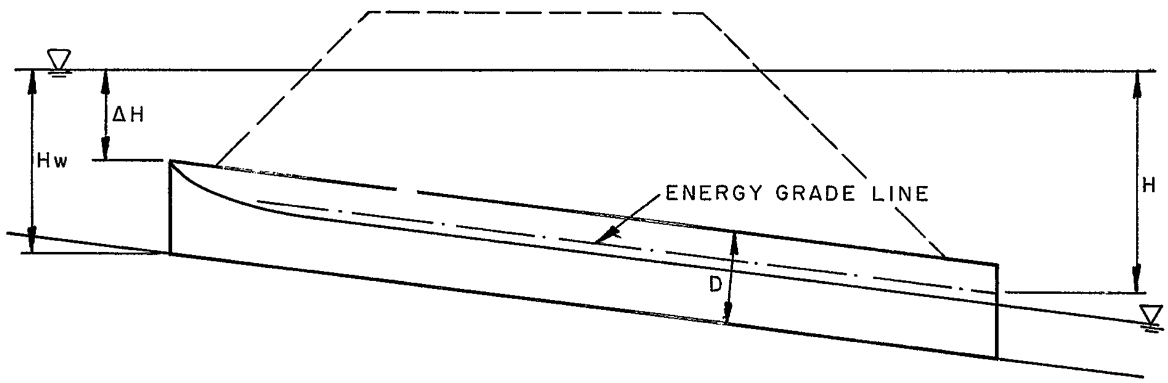


Figure 4—Inlet Control—Submerged Inlet

For a culvert operating with inlet control, the roughness, slope, and length of the culvert barrel and outlet conditions (including tailwater) are not factors in determining culvert hydraulic performance.

2.1.3 Outlet Control

If the headwater is high enough, the culvert slope sufficiently flat, and the culvert sufficiently long, the control will shift to the outlet. In outlet control, the discharge is a function of the inlet losses, the headwater depth, the culvert roughness, the culvert length, the barrel diameter, the culvert slope, and sometimes the tailwater elevation.

In outlet control, culvert hydraulic performance is determined by these factors:

- Depth of headwater
- Cross-sectional area
- Inlet edge configuration
- Culvert shape
- Barrel slope
- Barrel length
- Barrel roughness
- Depth of tailwater

Outlet control will exist under two conditions. The first and least common is that where the headwater is insufficient to submerge the top of the culvert, and the culvert slope is subcritical (Figure 5). The most common condition exists when the culvert is flowing full (Figure 6). A culvert flowing under outlet control is defined as a hydraulically long culvert.

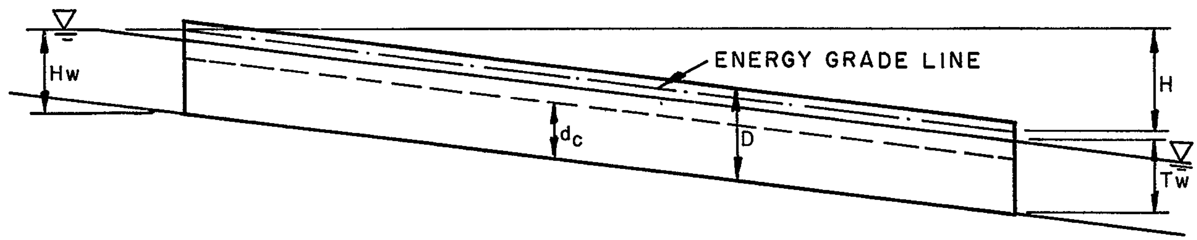


Figure 5—Outlet Control—Partially Full Conduit

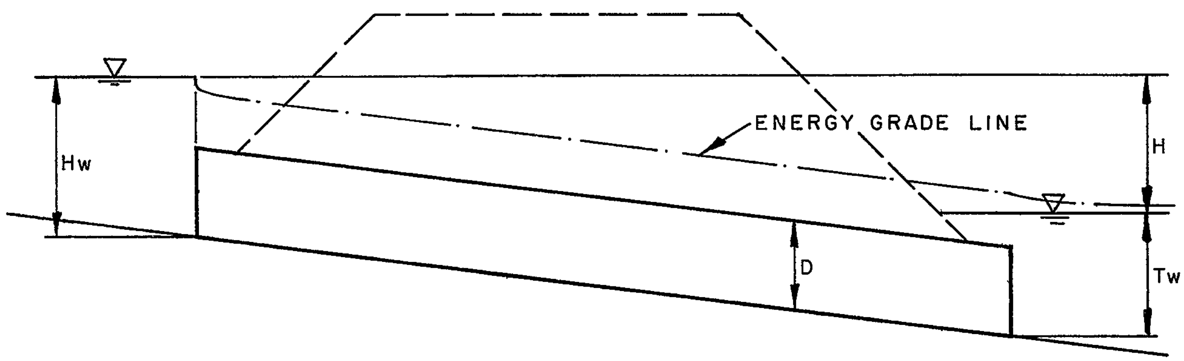


Figure 6—Outlet Control—Full Conduit

Culverts operating under outlet control may flow full or partly full depending on various combinations of the above factors. In outlet control, factors that may affect performance appreciably for a given culvert size and headwater are barrel length and roughness, and tailwater depth.

2.2 Energy Losses

In short conduits, such as culverts, the form losses due to the entrance can be as important as the friction losses through the conduit. The losses that must be evaluated to determine the carrying capacity of the culverts consist of inlet (or entrance) losses, friction losses, and outlet (or exit) losses.

2.2.1 Inlet Losses

For inlet losses, the governing equations are:

$$Q = CA\sqrt{2gH}$$

(Equation 4)

$$H_e = K_e \frac{v^2}{2g}$$

(Equation 5)

where:

Q = flow rate or discharge (cfs)

C = contraction coefficient (dimensionless)

A = cross-sectional area (ft^2)

g = acceleration due to gravity, 32.2 (ft/sec^2)

H = total head (ft)

H_e = head loss at entrance (ft)

K_e = entrance loss coefficient

v = average velocity (ft/sec)

2.2.2 Outlet Losses

For outlet losses, the governing equations are related to the difference in velocity head between the pipe flow and that in the downstream channel at the end of the pipe.

2.2.3 Friction Losses

Friction head loss for turbulent flow in pipes flowing full can be determined from the Darcy-Weisbach equation.

$$H_f = f\left(\frac{L}{D}\right)\left(\frac{v^2}{2g}\right)$$

(Equation 6)

where:

H_f = frictional head loss (ft)

f = friction factor (dimensionless)

L = length of culvert (ft)

D = Diameter of culvert (ft)

v = average velocity (ft/sec)

g = acceleration due to gravity, 32.2 (ft/sec^2)

The friction factor has been determined empirically and is dependent on relative roughness, velocity, and barrel diameter. Moody diagrams can be used to determine the friction factor. The friction losses for culverts are often expressed in terms of Manning's n, which is independent of the size of pipe and depth of flow. Another common formula for pipe flow is the Hazen-Williams formula. Standard hydraulic texts should be consulted for limitations of these formulas.

3.0 CULVERT SIZING AND DESIGN

FHWA (1985) Hydraulic Design Series No. 5, *Hydraulic Design of Highway Culvert*s, provides valuable guidance for the design and selection of drainage culverts. This circular explains inlet and outlet control and the procedure for designing culverts. Culvert design basically involves the trial and error method:

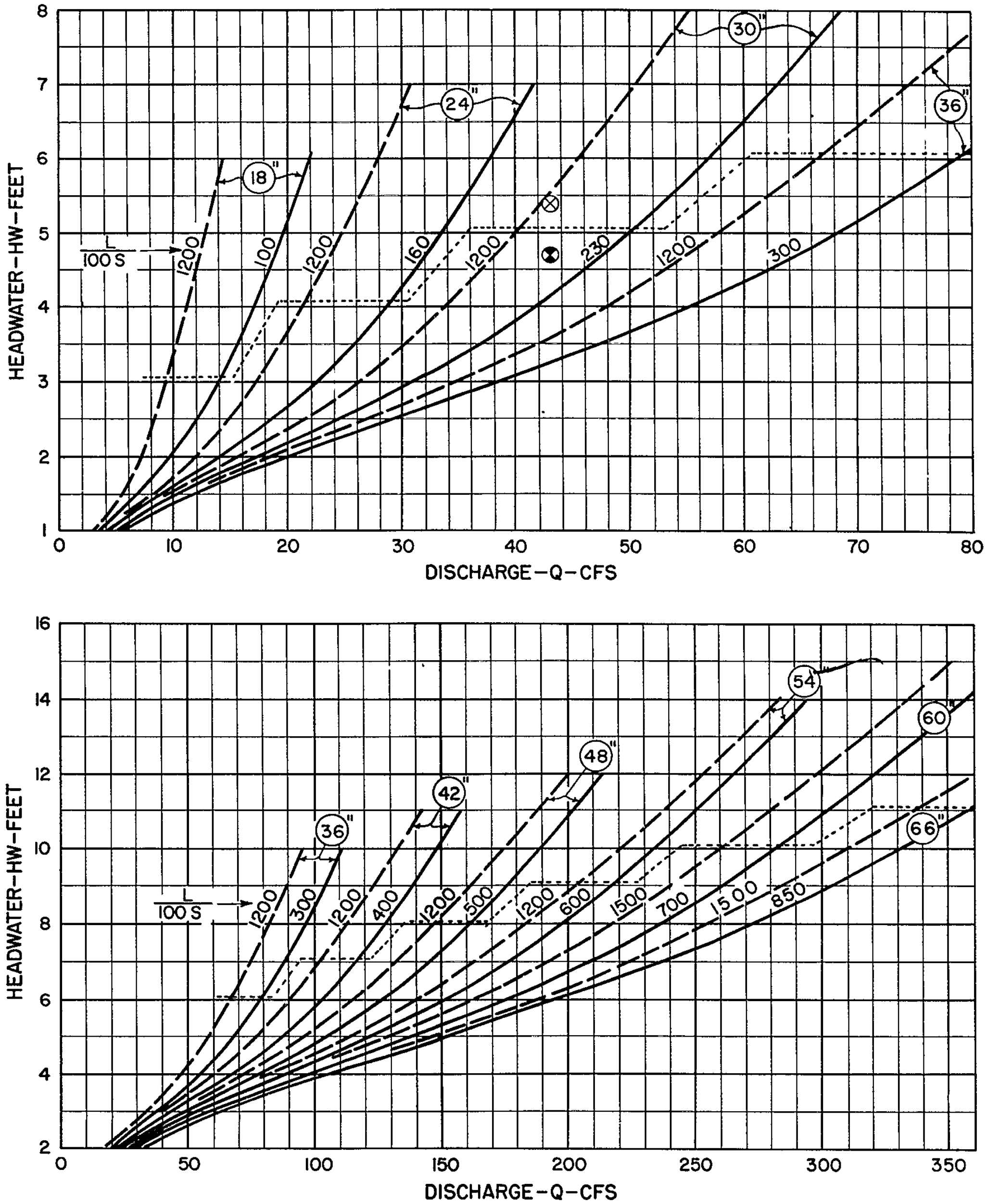
1. Select a culvert shape, type, and size with a particular inlet end treatment.
2. Determine a headwater depth from the relevant charts for both inlet and outlet control for the design discharge, the grade and length of culvert, and the depth of water at the outlet (tailwater).
3. Compare the largest depth of headwater (as determined from either inlet or outlet control) to the design criteria. If the design criteria are not met, continue trying other culvert configurations until one or more configurations are found to satisfy the design parameters.
4. Estimate the culvert outlet velocity and determine if there is a need for any special features such as energy dissipators, riprap protection, fish passage, trash/safety rack, etc.

3.1 Description of Capacity Charts

Figure 7 is an example of a capacity chart used to determine culvert size. Refer to this figure in the following discussion.

Each chart contains a series of curves, which show the discharge capacity per barrel in cfs for each of several sizes of similar culvert types for various headwater depths in feet above the invert of the culvert at the inlet. The invert of the culvert is defined as the low point of its cross section.

Each size is described by two lines, one solid and one dashed. The numbers associated with each line are the ratio of the length, L, in feet, to 100 times the slope, s, in feet per foot (ft/ft) (100s). The dashed lines represent the maximum $L/(100s)$ ratio for which the curves may be used without modification. The solid line represents the division between outlet and inlet control. For values of $L/(100s)$ less than that shown on the solid line, the culvert is operating under inlet control and the headwater depth is determined from the $L/(100s)$ value given on the solid line. The solid-line inlet-control curves are plotted from model test data. The dashed-line outlet-control curves were computed for culverts of various lengths with relatively flat slopes. Free outfall at the outlet was assumed; therefore, tailwater depth is assumed not to influence the culvert performance.



From BPR

EXAMPLE

⊗ GIVEN:
43 CFS ; AHW = 5.4 FT.
L = 120 FT.; S_0 = 0.002

SELECT 30"
HW = 4.7 FT.

Figure 7—Culvert Capacity Chart—Example

For culverts flowing under outlet control, the head loss at the entrance was computed using the loss coefficients previously given, and the hydraulic roughness of the various materials used in culvert construction was taken into account in computing resistance loss for full or part-full flow. The Manning's n values used for each culvert type ranged from 0.012 to 0.032.

Except for large pipe sizes, headwater depths on the charts extend to 3 times the culvert height. Pipe arches and oval pipe show headwater up to 2.5 times their height since they are used in low fills. The dotted line, stepped across the charts, shows headwater depths of about twice the barrel height and indicates the upper limit of restricted use of the charts. Above this line the headwater elevation should be checked with the nomographs, which are described in Section 3.3.

The headwater depth given by the charts is actually the difference in elevation between the culvert invert at the entrance and the total head; that is, depth plus velocity head for flow in the approach channel. In most cases, the water surface upstream from the inlet is so close to this same level that the chart determination may be used as headwater depth for practical design purposes. Where the approach velocity is in excess of 3.0 ft/sec, the velocity head must be subtracted from the curve determination of headwater to obtain the actual headwater depth.

3.2 Use of Capacity Charts

The procedure for sizing the culvert is summarized below. Data can be tabulated in the Design Computation Form shown in Figure 8.

1. List design data: Q = flow or discharge rate (cfs), L = length of culvert (ft), allowable Hw = headwater depth (ft), s = slope of culvert (ft/ft), type of culvert barrel, and entrance.

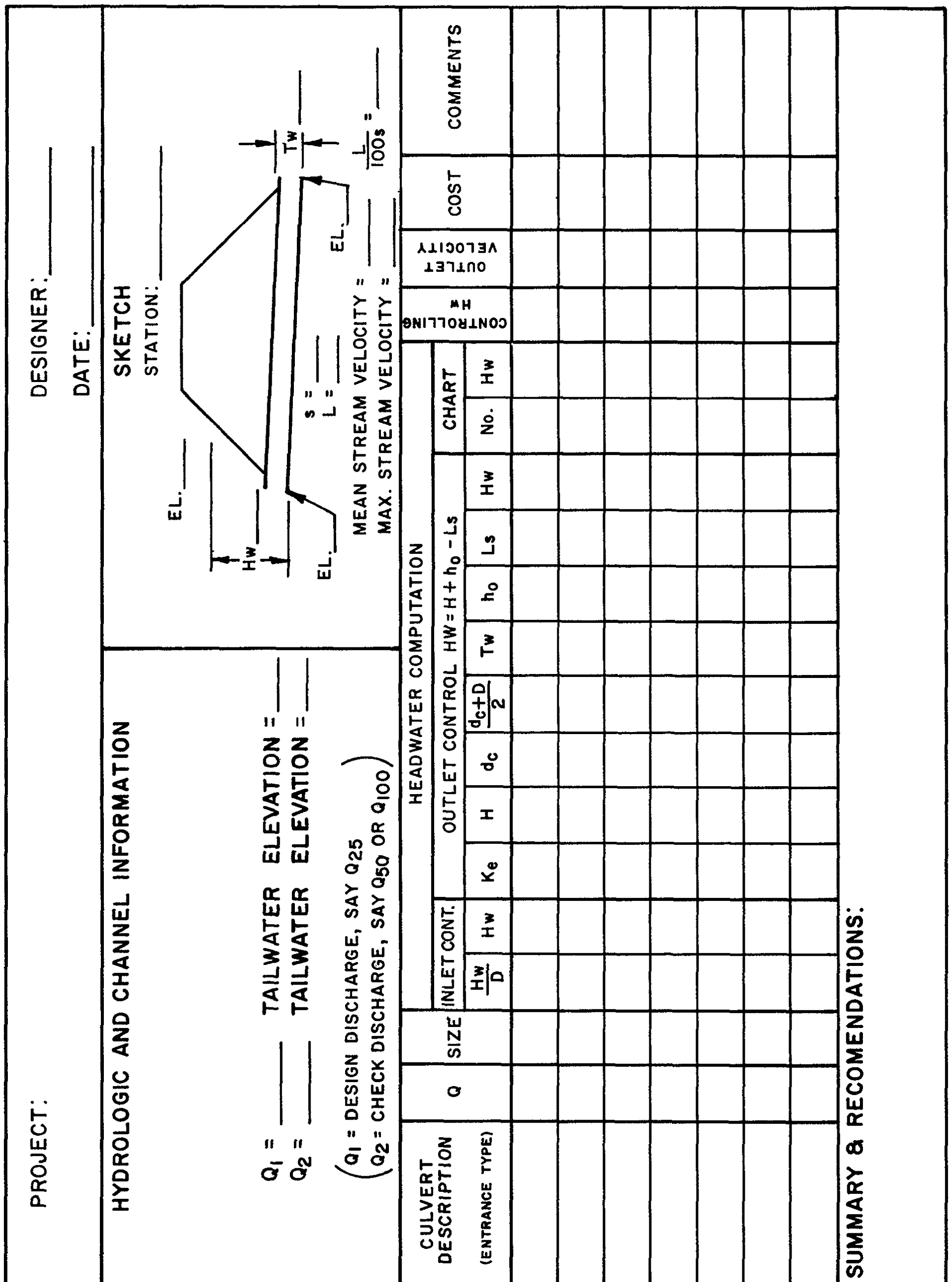
2. Compute $L/(100s)$.

3. Enter the appropriate capacity chart in Section 12.0 with the design discharge, Q.

4. Find the $L/(100s)$ value for the smallest pipe that will pass the design discharge. If this value is above the dotted line in Figure 7, use the nomographs to check headwater conditions.

5. If $L/(100s)$ is less than the value of $L/(100s)$ given for the solid line, then the value of Hw is the value obtained from the solid line curve. If $L/(100s)$ is larger than the value for the dashed outlet control curve, then special measures must be taken, and the reader is referred to *Hydraulic Design of Highway Culverts* (FHWA 1985).

6. Check the *Hw* value obtained from the charts with the allowable *Hw*. If the indicated *Hw* is greater than the allowable *Hw*, then try the *Hw* elevation from the next largest pipe size.



From BPR

Figure 8—Design Computation for Culverts—Blank Form

3.3 Use of Nomographs

Examples of two nomographs for designing culverts are presented in Figures 9 and 10. The use of these nomographs is limited to cases where tailwater depth is higher than the critical depth in the culvert. The advantage of the capacity charts over the nomographs is that the capacity charts are direct where the nomographs are trial and error. The capacity charts can be used only when the flow passes through critical depth at the outlet. When the critical depth at the outlet is less than the tailwater depth, the nomographs must be used; however, both give the same results where either of the two methods may be used. The procedure for design requires the use of both nomographs and is as follows (refer to Figures 9 and CU):

1. List design data: Q (cfs), L (ft), invert elevations in and out (ft), allowable Hw (ft), mean and maximum flood velocities in natural stream (ft/sec), culvert type and entrance type for first selection.

2. Determine a trial size by assuming a maximum average velocity based on channel considerations to compute the area, $A = Q/V$.

3. Find Hw for trial size culvert for inlet control and outlet control. For inlet control, Figure 9, connect a straight line through D and Q to scale (1) of the Hw/D scales and project horizontally to the proper scale, compute Hw and, if too large or too small, try another size before computing Hw for outlet control.

4. Next, compute the Hw for outlet control, Figure 10. Enter the graph with the length, the entrance coefficient for the entrance type, and the trial size. Connect the length scale and the culvert size scale with a straight line, pivot on the turning line, and draw a straight line from the design discharge on the discharge scale through the turning point to the head scale (head loss, H). Compute Hw from the equation:

$$Hw = H + h_o - Ls$$

(Equation 7)

where:

Hw = headwater depth (ft)

H = head loss (ft)

h_o = tailwater depth or elevation at the outlet of a depth equivalent to the location of the hydraulic grade line (ft)

L = length of culvert (ft)

s = slope of culvert (ft/ft)

For Tw greater than or equal to the top of the culvert, $h_o = Tw$, and for Tw less than the top of the culvert:

$$h_o = \frac{(d_c + D)}{2} \text{ or } T_w \text{ (whichever is greater)}$$

(Equation 8)

where:

dc = critical depth (ft)

Tw = tailwater depth (ft)

If Tw is less than d_c, the nomographs cannot be used, see *Hydraulic Design of Highway Culverts* (FHWA 1985) for critical depth charts.

Compare the computed headwaters and use the higher Hw to determine if the culvert is under inlet or outlet control. If outlet control governs and the Hw is unacceptable, select a larger trial size and find another Hw with the outlet control nomographs. Since the smaller size of culvert had been selected for allowable Hw by the inlet control nomographs, the inlet control for the larger pipe need not be checked.

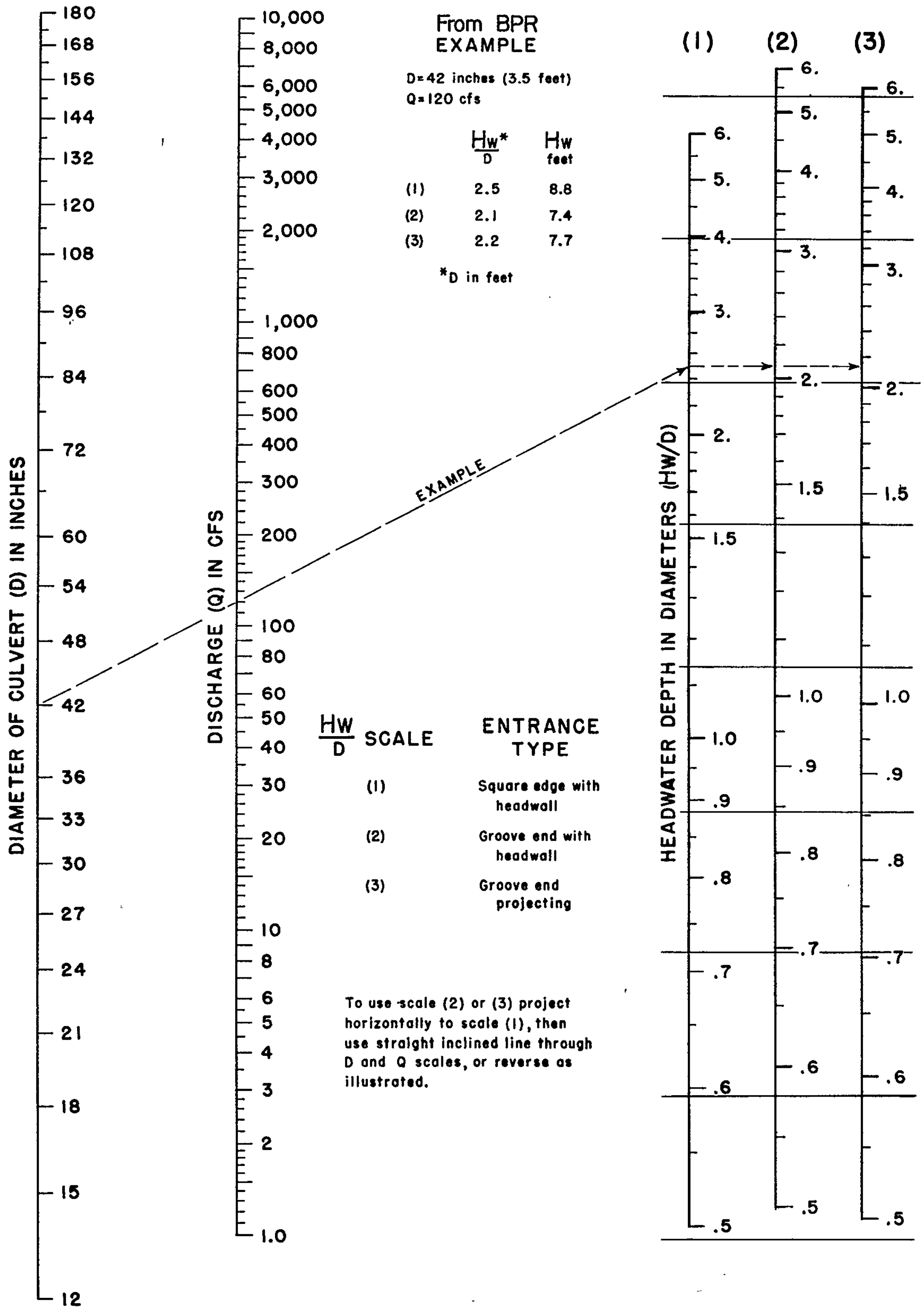


Figure 9—Inlet Control Nomograph—Example

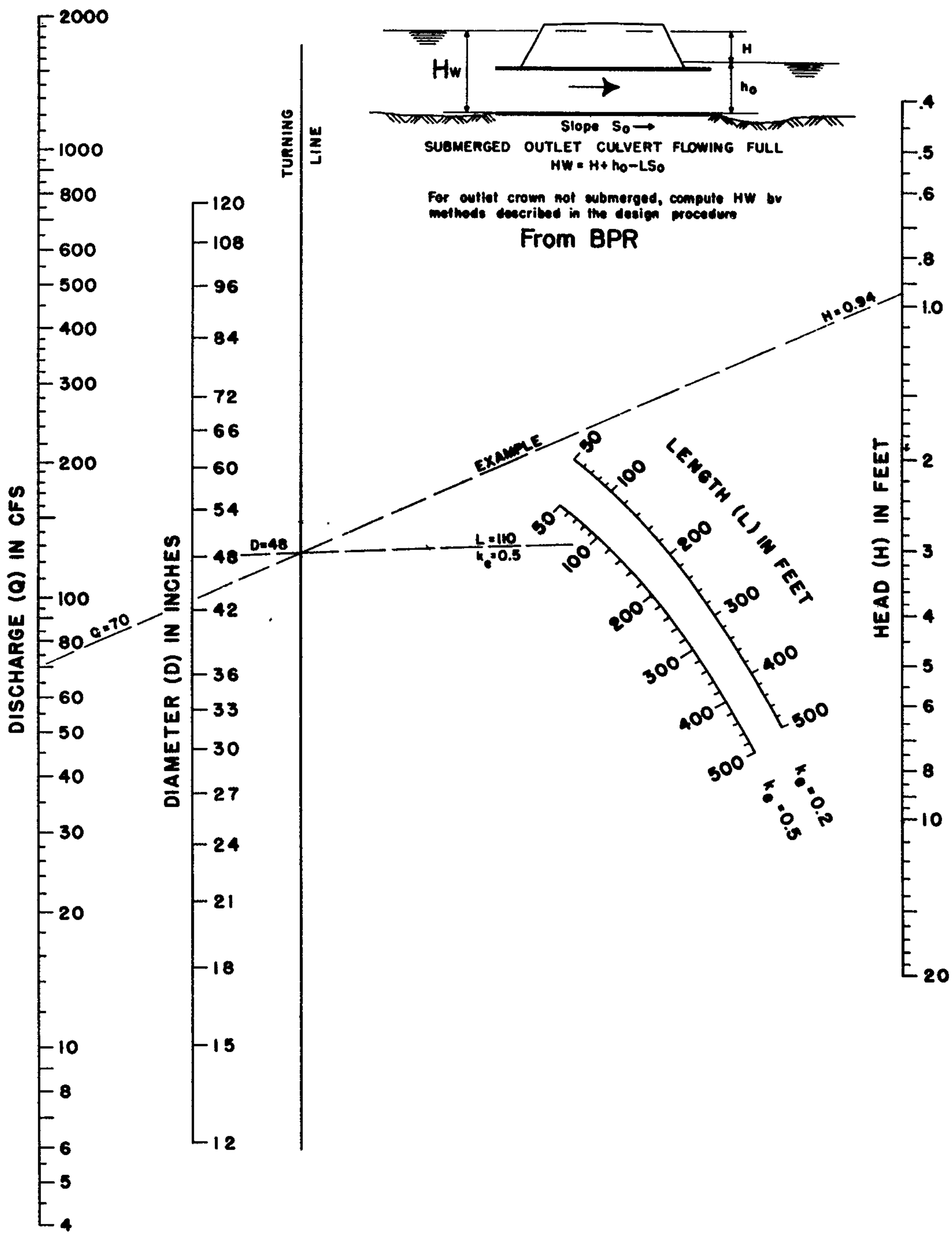


Figure 10—Outlet Control Nomograph—Example

3.4 Computer Applications, Including Design Spreadsheet

Although the nomographs discussed in this chapter are still used, engineers are increasingly designing culverts using computer applications. Among these applications are the FHWA's HY8 Culvert Analysis (Ginsberg 1987) and numerous proprietary applications. In addition, the District has developed spreadsheets to aid in the sizing and design of culverts. Both the UD-Culvert Spreadsheet application and FHWA's HY8 Culvert Analysis (Version 6.1) are located on the CD-ROM version of this *Manual*.

3.5 Design Considerations

Due to problems arising from topography and other considerations, the actual design of a culvert installation is more difficult than the simple process of sizing culverts. The information in the procedure for design that will be given is a guide to design since the problems encountered are too varied and too numerous to be generalized. However, the actual process presented should be followed to insure that some special problem is not overlooked. Several combinations of entrance types, invert elevations, and pipe diameters should be tried to determine the most economic design that will meet the conditions imposed by topography and engineering.

3.5.1 Design Computation Forms

The use of design computation forms is a convenient method to use to obtain consistent designs and promote cost-effectiveness. An example of such a form is Figure 8.

3.5.2 Invert Elevations

After determining the allowable headwater elevation, the tailwater elevation, and the approximate length, invert elevations must be assumed. Scour is not likely in an artificial channel such as a roadside ditch or a major drainage channel when the culvert has the same slope as the channel. To reduce the chance of failure due to scour, invert elevations corresponding to the natural grade should be used as a first trial. For natural channels, the flow conditions in the channel upstream from the culvert should be investigated to determine if scour will occur.

3.5.3 Culvert Diameter

After the invert elevations have been assumed and using the design computation forms (e.g., Figure 8), the capacity charts (e.g., Figure 7), and the nomographs, the diameter of pipe that will meet the headwater requirements should be determined. Since small diameter pipes are often plugged by sediment and debris, it is recommended that pipe smaller than 18 inches not be used for any drainage where this *Manual* applies. Since the pipe roughness influences the culvert diameter, both concrete and corrugated metal pipe should be considered in design, if both will satisfy the headwater requirements.

3.5.4 Limited Headwater

If there is insufficient headwater elevation to obtain the required discharge, it is necessary to oversize the culvert barrel, lower the inlet invert, use an irregular cross section, or use any combination of the preceding.

If the inlet invert is lowered, special consideration must be given to scour. The use of gabions or concrete drop structures, riprap, and headwalls with apron and toe walls should be investigated and compared to obtain a proper design.

3.6 Culvert Outlet

The outlet velocity must be checked to determine if significant scour will occur downstream during the major storm. If scour is indicated (and this will normally be the case), refer to Section 5.0 of this chapter ("Protection Downstream of Culverts") Inadequate culvert outlet protection is a common problem. Short-changing outlet protection is no place to economize during design and construction because downstream channel degradation can be significant and the culvert outlet can be undermined.

3.7 Minimum Slope

To minimize sediment deposition in the culvert, the culvert slope must be equal to or greater than the slope required to maintain a minimum velocity. The slope should be checked for each design, and if the proper minimum velocity is not obtained, the pipe diameter may be decreased, the slope steepened, a smoother pipe used, or a combination of these may be used.

4.0 CULVERT INLETS

A fact often overlooked is that a culvert cannot carry any more water than can enter the inlet. Frequently culverts and open channels are carefully designed with full consideration given to slope, cross section, and hydraulic roughness, but without regard to the inlet limitations. Culvert designs using uniform flow equations rarely carry their design capacity due to limitations imposed by the inlet.

The design of a culvert, including the inlet and the outlet, requires a balance between cost, hydraulic efficiency, purpose, and topography at the proposed culvert site. Where there is sufficient allowable headwater depth, a choice of inlets may not be critical, but where headwater depth is limited, where erosion is a problem, or where sedimentation is likely, a more efficient inlet may be required to obtain the necessary discharge for the culvert.

The primary purpose of a culvert is to convey flows. A culvert may also be used to restrict flow, that is, to discharge a controlled amount of water while the area upstream from the culvert is used for detention storage to reduce a storm runoff peak. For this case, an inefficient inlet may be the most desirable choice.

The inlet types described in this chapter may be selected to fulfill either of the above requirements depending on the topography or conditions imposed by the designer. The entrance coefficient, K_e, as defined by Equation 5, is a measure of the hydraulic efficiency at the inlet, with lower valves indicating greater efficiency. Inlet coefficients recommended for use are given in Table 1.

Table 1
Inlet Coefficients For Outlet Control

Type of Entrance	Entrance Coefficient, K_e
1. Pipe entrance with headwall	
Grooved edge	0.20
Rounded edge (0.15D radius)	0.15
Rounded edge (0.25D radius)	0.10
Square edge (cut concrete and CMP)	0.40
2. Pipe entrance with headwall & 45° wingwall	
Grooved edge	0.20
Square edge	0.35
3. Headwall with parallel wingwalls spaced 1.25D apart	
Grooved edge	0.30
Square edge	0.40
4. Special inlets—see Section 4.3	
5. Projecting Entrance	
Grooved edge	0.25
Square edge	0.50
Sharp edge, thin wall	0.90

4.1 Projecting Inlets

Projecting inlets should not be used. Headwalls, wingwalls, and flared end sections should be used to maximize efficiency and minimize turbulence, head loss, and erosion.

4.2 Inlets with Headwalls

Headwalls may be used for a variety of reasons, including increasing the efficiency of the inlet, providing embankment stability, and providing embankment protection against erosion. The relative efficiency of the inlet varies with the pipe material used. Figure 12 illustrates a headwall with wingwalls.

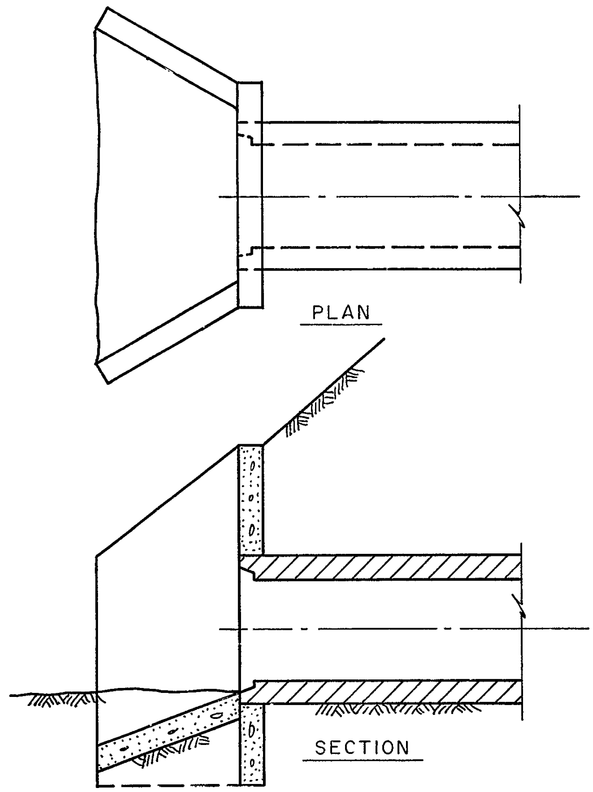


Figure 11—Inlet With Headwall and Wingwalls

4.2.1 Corrugated Metal Pipe

Corrugated metal pipe in a headwall is essentially a square-edged entrance with an entrance coefficient of about 0.4. The entrance losses may be reduced by rounding the entrance. The entrance coefficient may be reduced to 0.15 for a rounded edge with a radius equal to 0.15 times the culvert diameter, and to 0.10 for rounded edge with a radius equal to 0.25 times the diameter of the culvert.

4.2.2 Concrete Pipe

For tongue-and-groove or bell-end concrete pipe, little increase in hydraulic efficiency is realized by adding a headwall. The primary reason for using headwalls is for embankment protection and for ease of maintenance. The entrance coefficient is equal to about 0.2 for grooved and bell-end pipe, and equal to 0.4 for cut concrete pipe.

4.2.3 Wingwalls

Wingwalls are used where the side slopes of the channel adjacent to the entrance are unstable and where the culvert is skewed to the normal channel flow. Little increase in hydraulic efficiency is realized with the use of wingwalls, regardless of the pipe material used and, therefore, the use should be justified for reasons other than an increase in hydraulic efficiency. Figure 13 illustrates several cases where wingwalls are used. For parallel wingwalls, the minimum distance between wingwalls should be at least 1.25 times the diameter of the culvert pipe.

4.2.4 Aprons

If high headwater depths are to be encountered, or if the approach velocity of the channel will cause scour, a short channel apron should be provided at the toe of the headwall. This apron should extend at least one pipe diameter upstream from the entrance, and the top of the apron should not protrude above the normal streambed elevation.

Culverts with wingwalls should be designed with a concrete apron extending between the walls. Aprons must be reinforced to control cracking. As illustrated in Figure 13, the actual configuration of the wingwalls varies according to the direction of flow and will also vary according to the topographical requirement placed upon them.

For conditions where scour may be a problem due to high approach velocities and special soil conditions, such as alluvial soils, a toe wall is often desirable for apron construction.

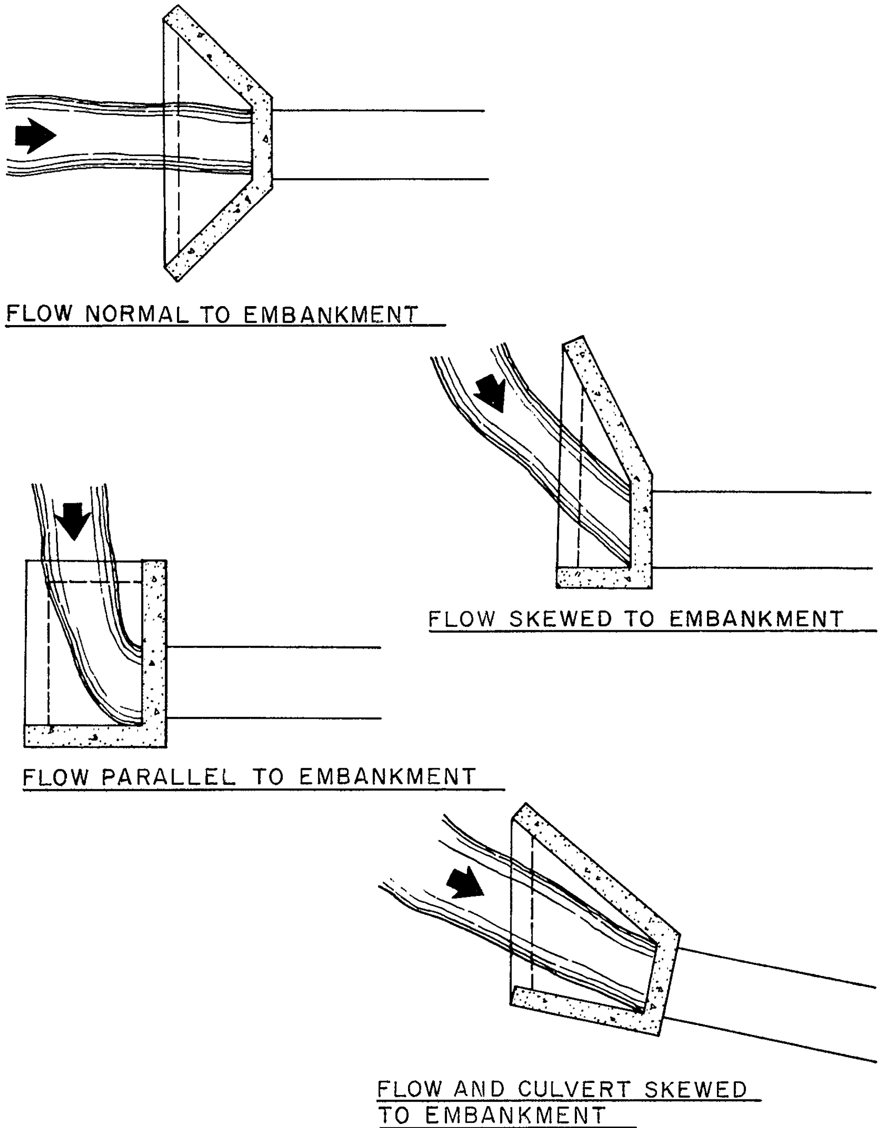


Figure 12—Typical Headwall-Wingwall Configurations

4.3 Special Inlets

There is a great variety of inlets other than the common ones described. Among these are special end-sections, which serve as both outlets and inlets and are available for both corrugated metal pipe and concrete pipe. Because of the difference in requirements due to pipe materials, the special end-sections will be discussed independently according to pipe material, and mitered inlets will also be considered.

4.3.1 Corrugated Metal Pipe

Special end-sections for corrugated metal pipe add little to the overall cost of the culvert and have the following advantages:

1. Less maintenance around the inlet.
2. Less damage from maintenance work and from accidents compared to a projecting entrance.
3. Increased hydraulic efficiency. When using design charts, as discussed in Section 3.0, charts for a square-edged opening for corrugated metal pipe with a headwall may be used.

4.3.2 Concrete Pipe

As in the case of corrugated metal pipe, these special end-sections may aid in increasing the embankment stability or in retarding erosion at the inlet. They should be used where maintenance equipment must be used near the inlet or where, for aesthetic reasons, a projecting entrance is considered too unsightly.

The hydraulic efficiency of this type of inlet is dependent on the geometry of the end-section to be used. Where the full contraction to the culvert diameter takes place at the first pipe section, the entrance coefficient, K_e, is equal to 0.5, and where the full contraction to the culvert diameter takes place in the throat of the end-section, the entrance coefficient, K_e, is equal to 0.25.

4.3.3 Mitered Inlets

The use of this entrance type is predominantly with corrugated metal pipe and its hydraulic efficiency is dependent on the construction procedure used. If the embankment is not paved, the entrance, in practice, usually does not conform to the side slopes, giving essentially a projecting entrance with K_e = 0.9. If the embankment is paved, a sloping headwall is obtained with K_e = 0.60 and, by beveling the edges, K_e = 0.50.

Uplift is an important factor for this type entrance. It is not good practice to use unpaved embankment slopes where a mitered entrance may be submerged to an elevation one-half the diameter of the culvert above the top of the pipe.

4.3.4 Long Conduit Inlets

Inlets are important in the design of culverts for road crossings and other short sections of conduit; however, they are even more significant in the economical design of long culverts and pipes. Unused capacity in a long conduit will result in wasted investment. Long conduits are costly and require detailed engineering, planning, and design work. The inlets to such conduits are extremely important to the functioning of the conduit and must receive special attention.

Most long conduits require special inlet considerations to meet the particular hydraulic characteristics of the conduit. Generally, on larger conduits, hydraulic model testing will result in better and less costly inlet construction.

4.4 Improved Inlets

Inlet edge configuration is one of the prime factors influencing the performance of a culvert operating under inlet control. Inlet edges can cause a severe contraction of the flow, as in the case of a thin edge, projecting inlet. In a flow contraction, the effective cross-sectional area of the barrel may be reduced to about one-half of the actual available barrel area. As the inlet configuration is improved, the flow contraction is reduced, thus improving the performance of the culvert.

A tapered inlet is a flared culvert inlet with an enlarged face section and a hydraulically efficient throat section. Tapered inlets improve culvert performance by providing a more efficient control section (the throat). However, tapered inlets are not recommended for use on culverts flowing under outlet control because the simple beveled edge is of equal benefit. The two most common improved inlets are the side-tapered inlet and the slope-tapered inlet (Figure 14). FHWA (1985) *Hydraulic Design of Highway Culverts* provides guidance on the design of improved inlets.

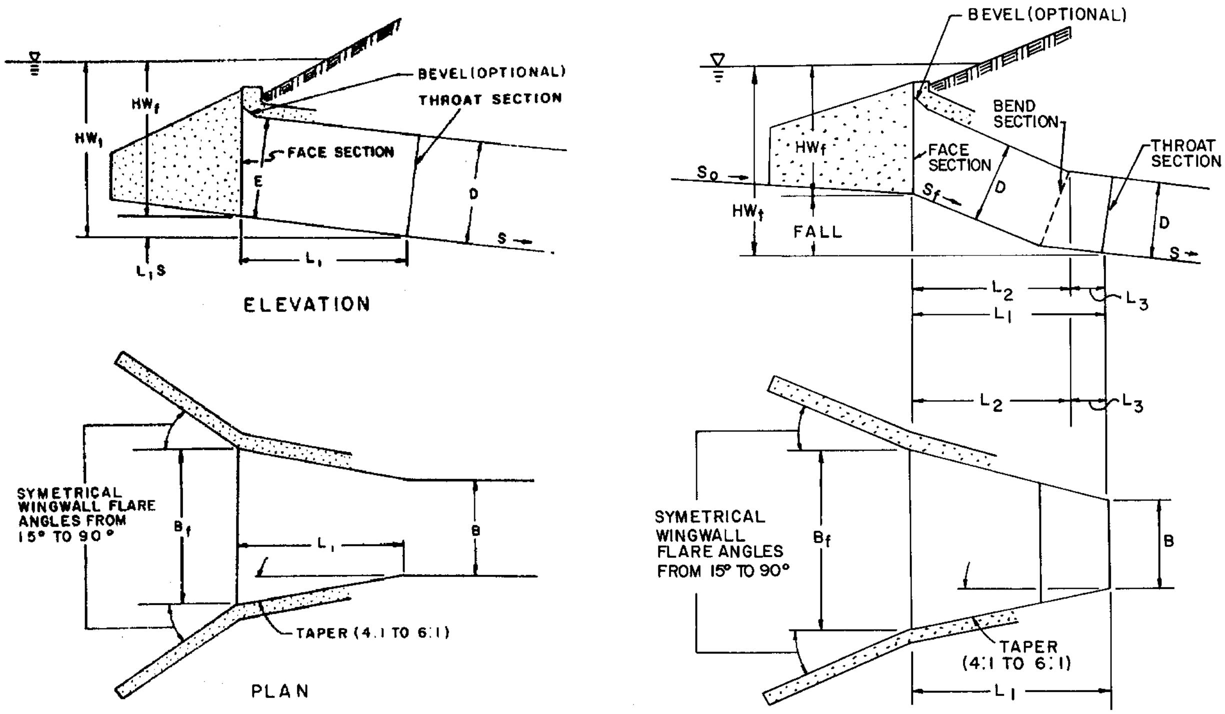


Figure 13—Side-Tapered and Slope-Tapered Improved Inlets

4.5 Inlet Protection

Inlets on culverts, especially on culverts to be installed in live streams, should be evaluated relative to debris control and buoyancy.

4.5.1 Debris Control

Accumulation of debris at a culvert inlet can result in the culvert not performing as designed. The consequences may be damages from inundation of the road and upstream property. The designer has three options for coping with the debris problem:

1. Retain the debris upstream of the culvert.
2. Attempt to pass the debris through the culvert.
3. Install a bridge.

If the debris is to be retained by an upstream structure or at the culvert inlet, frequent maintenance may be required. The design of a debris control structure should include a thorough study of the debris problem.

The following are among the factors to be considered in a debris study:

- Type of debris
- Quantity of debris
- Expected changes in type and quantity of debris due to future land use
- Stream flow velocity in the vicinity of culvert entrance
- Maintenance access requirements
- Availability of storage
- Maintenance plan for debris removal
- Assessment of damage due to debris clogging, if protection is not provided

Hydraulic Engineering Circular No. 9, *Debris Control Structure*s (FHWA 1971), should be used when designing debris control structures.

4.5.2 Buoyancy

The forces acting on a culvert inlet during flows are variable and indeterminate. When a culvert is functioning with inlet control, an air pocket begins just inside the inlet that creates a buoyant effect when the inlet is submerged. The buoyancy forces increase with an increase in headwater depth under inlet control conditions. These forces, along with vortexes and eddy currents, can cause scour, undermine culvert inlets, and erode embankment slopes, thereby making the inlet vulnerable to failure, especially with deep headwater.

In general, installing a culvert in a natural stream channel constricts the normal flow. The constriction is accentuated when the capacity of the culvert is impaired by debris or damage.

The large unequal pressures resulting from inlet constriction are in effect buoyant forces that can cause entrance failures, particularly on corrugated metal pipe with mitered, skewed, or projecting ends. The failure potential will increase with steepness of the culvert slope, depth of the potential headwater, flatness of the fill slope over the upstream end of the culvert, and the depth of the fill over the pipe.

Anchorage at the culvert entrance helps to protect against these failures by increasing the deadload on the end of the culvert, protecting against bending damage, and by protecting the fill slope from the scouring action of the flow. Providing a standard concrete headwall or endwall helps to counteract the hydrostatic uplift and to prevent failure due to buoyancy.

Because of a combination of high head on the outside of the inlet and the large region of low pressure on the inside of the inlet due to separation, a large bending moment is exerted on the end of the culvert, which may result in failure. This problem has been noted in the case of culverts under high fills, on steep slopes, and with projecting inlets. Where upstream detention storage requires headwater depth in excess of 20 feet, reducing the culvert size is recommended rather than using the inefficient projecting inlet to reduce discharge.

4.6 Trash/Safety Racks

The use of typical gratings at inlets to culverts and long underground pipes should be considered on a case-by-case basis. While there is a sound argument for the use of gratings for safety reasons, field experience has clearly shown that when the culvert is needed the most, that is, during the heavy runoff, normal gratings often become clogged and the culvert is rendered ineffective. A general rule of thumb is that if it will be feasible to “see daylight” from one side of the culvert to the other, a trash/safety rack will not be needed. By contrast, at entrances to longer culverts and long underground pipes, a trash rack is necessary.

The trash/safety rack design process is a matter of working out the safety hazard aspects of the problem with care, defining them clearly, and then taking reasonable steps to minimize safety hazards, yet protecting the integrity of the water carrying capability of the culvert (Photograph 5).

Photograph 5—Small trash racks at culvert entrance will increase the risk of entrance plugging.

Generally, the most common aspect to consider in evaluating the safety hazard of a culvert or long underground pipe opening is the possibility of a person, especially children, being carried into the culvert by flowing water approaching the inlet. In reviewing hazards, it is necessary to consider depth and velocity of upstream flow, the fact that storm runoff occurs during unusually heavy rain storms when most people, particularly children, are indoors, the general character of the neighborhood upstream, the length and size of culvert, and other similar factors. Furthermore, in the event that someone was carried to the culvert with the storm runoff, the exposure hazard may in some cases be even greater if the person is pinned to the grating by the hydrostatic pressure of the water rather than being carried through the culvert. Large, sloped racks positioned well in front of the culvert entrance reduce the risk of pinning.

Where debris potential and/or public safety indicate that a rack is required, if the pipe diameter is more than 24 inches, the rack's open surface area must be, at the absolute minimum, at least five times larger. For smaller pipes, the factor increases significantly. For culverts larger than 24 inches (i.e., in the smallest dimension), in addition to the trash rack having an open area larger than five times the culvert entrance, the average velocities at the rack's face shall be less than 2.0 feet per second at every stage of flow entering the culvert. The rack needs to be sloped no steeper than 3H:IV (the flatter the better) and have a clear opening at the bottom of 9 to 12 inches to permit debris at lower flows to go through. The bars on the face of the rack should be generally paralleling the flow and be spaced to provide 4 ½ to 5-inch clear openings between them. Transverse support bars need to be as few as possible, but sufficient to keep the rack from collapsing under full hydrostatic loads.

The installation of trash racks at culvert outlets is strongly discouraged because debris or a person carried into the culvert will impinge against the rack, thus leading to pressurized conditions within the culvert, virtually destroying its flow capacity and creating a greater hazard to the public or a person trapped in the culvert than not having one.

4.6.1 Collapsible Gratings

The City of Springfield does not recommend the use of collapsible gratings. On larger culverts where gratings are found to be necessary, the use of collapsible gratings may be desirable. Such gratings must be carefully designed from the structural standpoint so that collapse is achieved with a hydrostatic load of perhaps one-half of the maximum backwater head allowable. Collapse of the trash rack should be such that it clears the waterway opening adequately to permit the inlet to function properly.

4.6.2 Upstream Trash Collectors

In lieu of a collapsible trash rack and where a safety hazard exists, a grating situated a reasonable distance upstream from the actual inlet is often satisfactory. This type of grating may be a series of vertical pipes or posts embedded in the approach channel bottom. If blocking of this grating occurs, the backwater effect causes water to flow over the top of the grating and into the culvert with only minimal upstream backwater effect. The grating must not be so high as to cause the water to rise higher than the maximum allowable elevation.

5.0 PROTECTION DOWNSTREAM OF CULVERTS

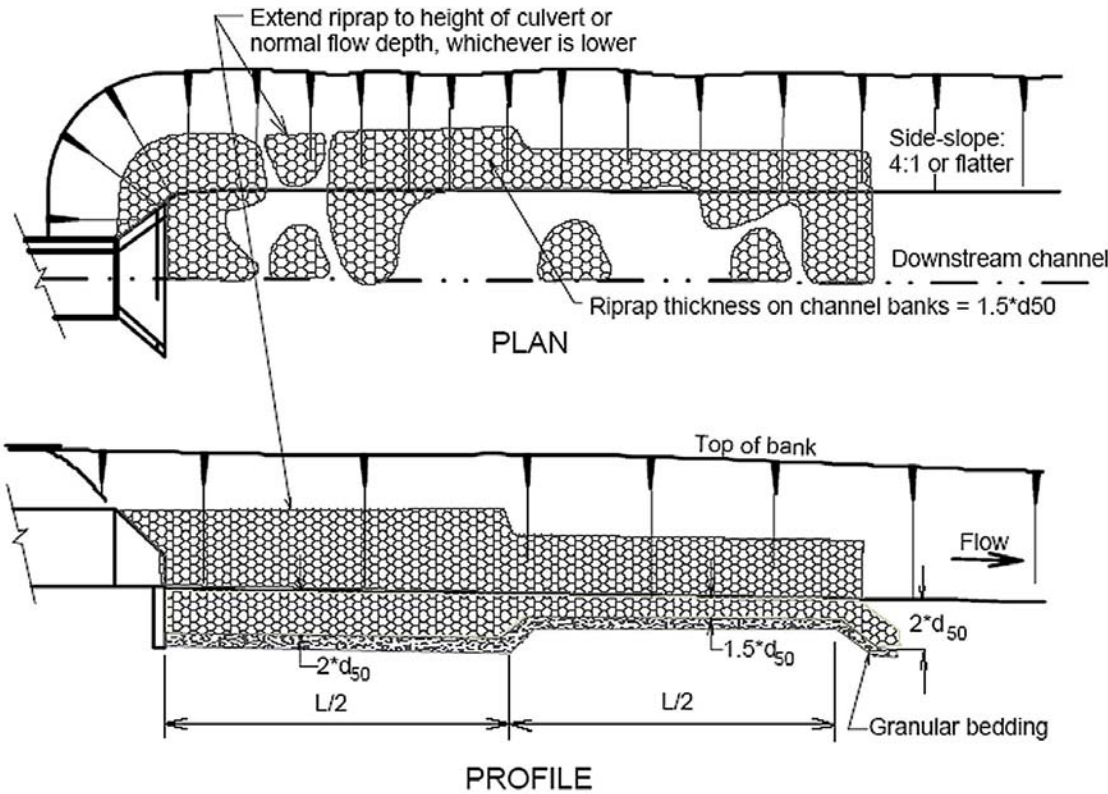
5.1 In-Line Riprap Protection

This section addresses the use of riprap for erosion protection downstream of conduit and culvert outlets ***in-line*** with major drainageway channels. The following section addresses ***energy dissipation structures***.

Scour resulting from highly turbulent, rapidly decelerating flow is a common problem at conduit outlets. The following riprap protection suggested below is for conduit and culvert outlet Froude numbers up to 2.5 (i.e., Froude parameters $Q/d_0^{2.5}$ or $Q/WH^{1.5}$ up to 14 $ft^{0.5}/sec$) where the conduit slopes are parallel with the channel gradient and the conduit outlet invert is flush with the riprap channel protection. Here, Q is the discharge in cfs, d_0 is the diameter of a circular conduit in feet and W and H are the width and height, respectively, of a rectangular conduit in feet.

5.1.1 Configuration of Riprap Protection

Figure 14 illustrates typical riprap protection of culverts and major drainageway conduit outlets. The additional thickness of the riprap just downstream from the outlet is to assure protection from flow conditions that might precipitate rock movement in this region.



NOTES: 1. Headwall with wingwalls or flared end section required at all culvert outlets.
2. Cutoff wall required at end of wingwall aprons and end section.
Minimum depth of cutoff wall = 2*d50 or 3-feet, whichever is deeper.
3. Provide joint fasteners for flared end sections.

Figure 14—Culvert and Pipe Outlet Erosion Protection

5.1.2 Required Rock Size

The required rock size may be selected from Figure 15 for circular conduits and from Figure 16 for rectangular conduits. Figure 15 is valid for $Q/D_c^{2.5}$ of 6 or less and Figure 16 is valid for $Q/WH^{1.5}$ of 8.0 or less. The parameters in these two figures are:

1. $Q/D^{1.5}$ or $Q/WH^{0.5}$ in which Q is the design discharge in cfs, D_c is the diameter of a circular conduit in feet, and W and H are the width and height of a rectangular conduit in feet.

2. Y_t/D_c or Y_t/H in which Y_t is the tailwater depth in feet, D_c is the diameter of a circular conduit in feet, and H is the height of a rectangular conduit in feet. In cases where Y_t is unknown or a hydraulic jump is suspected downstream of the outlet, use $Y_t/D_t = Y_t/H = 0.40$ when using Figures 15 and 16.

3. The riprap size requirements in Figures 14 and 15 are based on the non-dimensional parametric Equations 9 and 10 (Steven, Simons, and Lewis 1971 and Smith 1975).

   Circular culvert:

$$\frac{\left(\frac{d_{50}}{D_c}\right)\left(\frac{Y_t}{D_c}\right)^{1.2}}{\left(\frac{Q}{D_c^{2.5}}\right)} = 0.023$$

(Equation 9)

Rectangular culvert:

$$\frac{\left(\frac{d_{50}}{H}\right)\left(\frac{Y_t}{H}\right)}{\left(\frac{Q}{WH^{1.5}}\right)} = 0.014$$

(Equation 10)

The rock size requirements were determined assuming that the flow in the culvert barrel is not supercritical. It is possible to use Equations 9 and 10 when the flow in the culvert is supercritical (and less than full) if the value of D_c or H is modified for use in Figures 16 and 17. Whenever the flow is supercritical in the culvert, substitute D_a for D_c and H_a for H, in which D_a is defined as:

$$D_a = \frac{(D_c + Y_n)}{2}$$

(Equation 11)

in which the maximum value of D_a shall not exceed D, and

$$H_a = \frac{(H + Y_n)}{2}$$

(Equation 12)

in which the maximum value of H_a shall not exceed H, and:

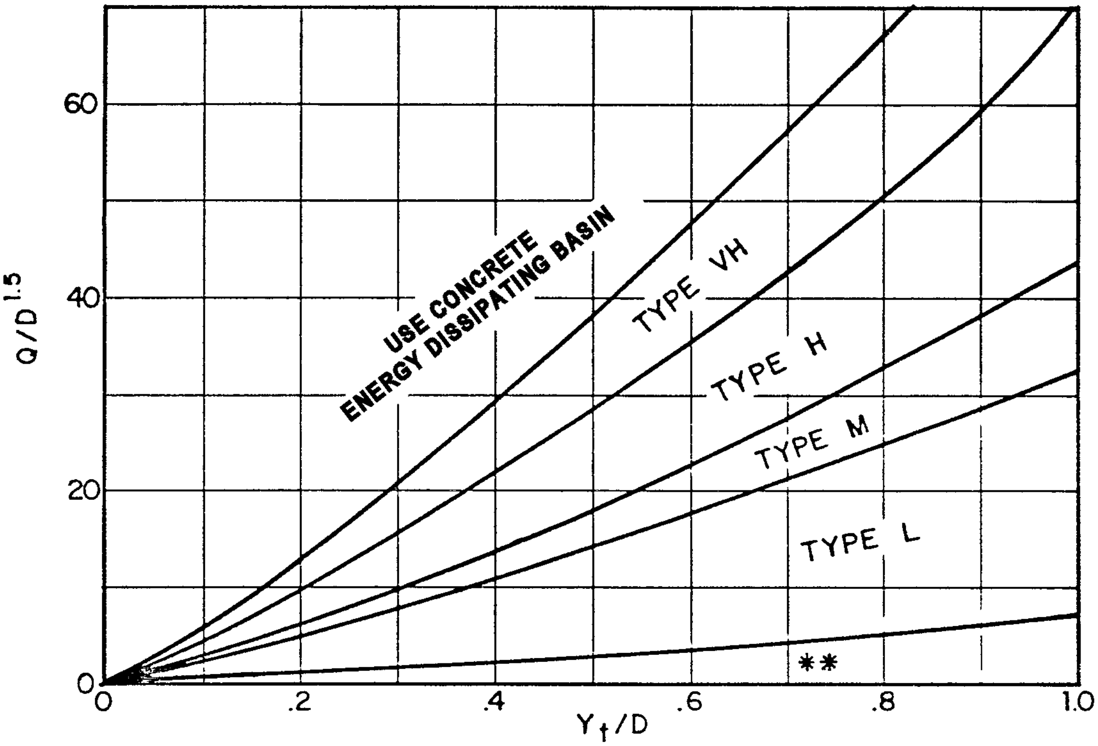
D_a = parameter to use in place of D in Figure 15 when flow is supercritical

D_c = diameter of circular culvert (ft)

H_a = parameter to use in place of H in Figure 16 when flow is supercritical

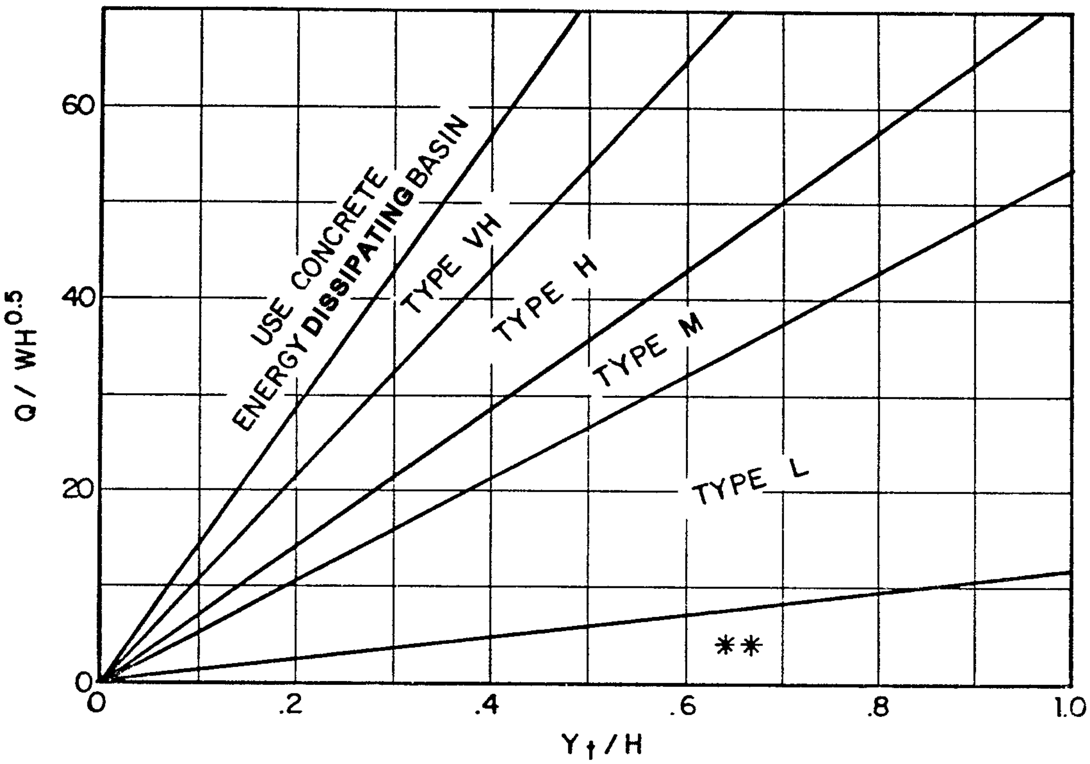
H = height of rectangular culvert (ft)

Y_n = normal depth of supercritical flow in the culvert



Use D_a instead of D whenever flow is supercritical in the barrel.
** Use Type L for a distance of 3D downstream.

Figure 15—Riprap Erosion Protection at Circular Conduit Outlet
Valid for $Q/D^{2.5} \leq 6.0$



Use H_a instead of H whenever culvert has supercritical flow in the barrel.
**Use Type L for a distance of 3H downstream.

Figure 16—Riprap Erosion Protection at Rectangular Conduit Outlet
Valid for $Q/WH^{1.5} \leq 8.0$

5.1.3 Extent of Protection

The length of the riprap protection downstream from the outlet depends on the degree of protection desired. If it is necessary to prevent all erosion, the riprap must be continued until the velocity has been reduced to an acceptable value. For purposes of outlet protection during major floods, the acceptable velocity is set at 5.5 ft/sec for very erosive soils and at 7.7 ft/sec for erosion resistant soils. The rate at which the velocity of a jet from a conduit outlet decreases is not well known. For the procedure recommended here, it is assumed to be related to the angle of lateral expansion, θ, of the jet. The velocity is related to the expansion factor, $(1/(2\tan\theta))$, which can be determined directly using Figure 433 or Figure 434, assuming that the expanding jet has a rectangular shape:

$$L_p = \left(\frac{1}{2\tan\theta}\right)\left(\frac{A_t}{Y_t} - W\right)$$

(Equation 13)

where:

L_p = length of protection (ft)

W = width of the conduit in (ft) (use diameter for circular conduits)

Y_t = tailwater depth (ft)

θ = the expansion angle of the culvert flow

and:

$$A_t = \frac{Q}{V}$$

(Equation 14)

where:

Q = design discharge (cfs)

V = the allowable non-eroding velocity in the downstream channel (ft/sec)

A_t = required area of flow at allowable velocity (ft²)

In certain circumstances, Equation 13 may yield unreasonable results. Therefore, in no case should L_p be less than 3H or 3D, nor does L_p need to be greater than 10H or 10D whenever the Froude parameter, $Q/WH^{1.5}$ or $Q/D^{2.5}$, is less than 8.0 or 6.0, respectively. Whenever the Froude parameter is greater than these maximums, increase the maximum L_p required by ¼ D_c or ¼ H for circular or rectangular culverts, respectively, for each whole number by which the Froude parameter is greater than 8.0 or 6.0, respectively.

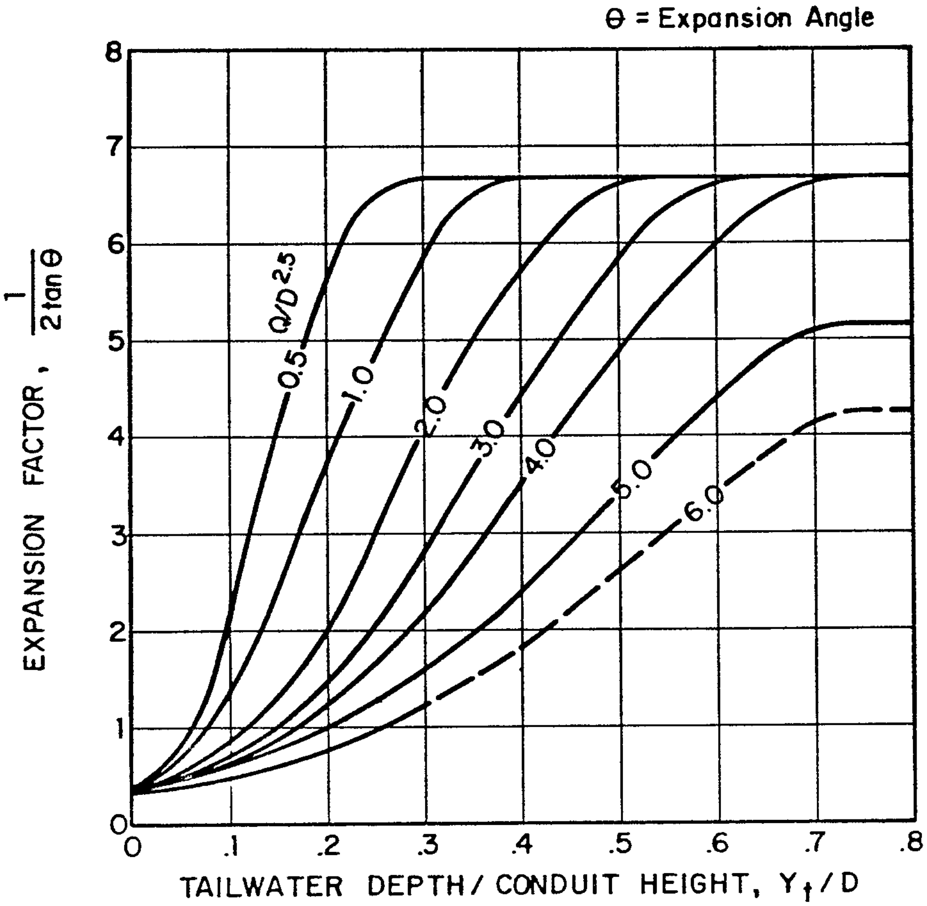


Figure 17—Expansion Factor for Circular Conduits

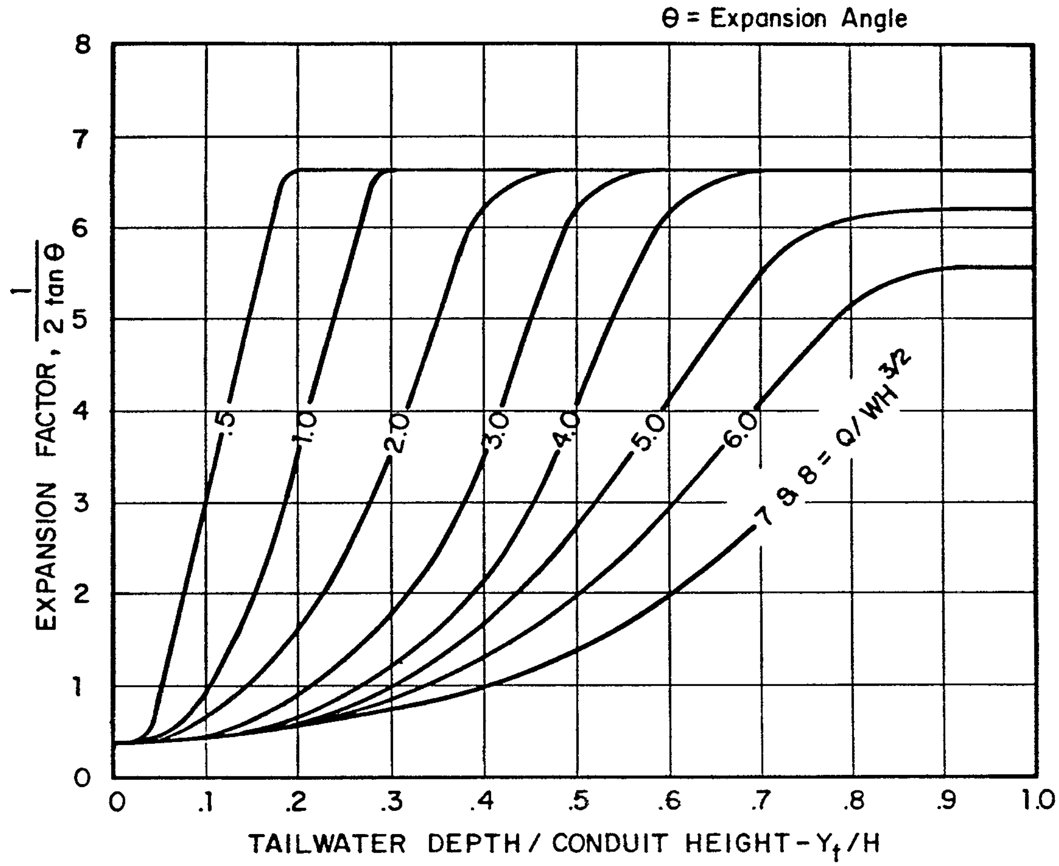


Figure 18—Expansion Factor for Rectangular Conduits

5.1.4 Multiple Conduit Installations

The procedures outlined in Sections 5.1.1, 5.1.2, and 5.1.3 can be used to design outlet erosion protection for multi-barrel culvert installations by hypothetically replacing the multiple barrels with a single hydraulically equivalent rectangular conduit. The dimensions of the equivalent conduit may be established as follows:

1. Distribute the total discharge, Q, among the individual conduits. Where all the conduits are hydraulically similar and identically situated, the flow can be assumed to be equally distributed; otherwise, the flow through each barrel must be computed.

2. Compute the Froude parameter $Q_i/D_{ci}^{2.5}$ (circular conduit) or $Q_i/W_iH_i^{1.5}$ (rectangular conduit), where the subscript i indicates the discharge and dimensions associated with an individual conduit.

3. If the installation includes dissimilar conduits, select the conduit with the largest value of the Froude parameter to determine the dimensions of the equivalent conduit.

4. Make the height of the equivalent conduit, H_{eq}, equal to the height, or diameter, of the selected individual conduit.

5. The width of the equivalent conduit, W_{eq}, is determined by equating the Froude parameter from the selected individual conduit with the Froude parameter associated with the equivalent conduit, $Q/W_iH_{eq}^{1.5}$.

5.2 Energy Dissipation Structures

Energy dissipation or stilling basin structures are required to minimize scour damages caused by high exit velocities and turbulence at conduit and culvert outlets. Outlet structures can provide a high degree of energy dissipation and are generally effective even with relatively low tailwater control. Rock protection downstream of conduit and culvert outlets (see the previous section) is appropriate where moderate outlet conditions exist; however, there are many situations where rock basins are impractical. Reinforced concrete outlet structures are suitable for a wide variety of site conditions. In some cases, they are more economical than larger rock basins, particularly when long-term costs are considered.

Any outlet structure must be designed to match the receiving stream conditions. The following steps include an analysis of the probable range of tailwater and bed conditions that can be anticipated including degradation, aggradation, and local scour.

Hydraulic concepts and design criteria are provided in this section for an impact stilling basin and adaptation of a baffle chute to conduit outlets. Use of concrete is often more economical due to structure size or local availability of materials. Initial design selection should include consideration of an energy dissipation structure if any of the following situations exist: (1) high-energy dissipation efficiency is required, where hydraulic conditions approach or exceed the limits for alternate designs (see previous section); (2) low tailwater control is anticipated; or (3) site conditions, such as public use areas, where plunge pools and standing water are unacceptable because of safety and appearance, or at locations where space limitations direct the use of a concrete structure.

Longer conduits with large cross-sectional areas are designed for significant discharges and often with high velocities requiring special hydraulic design at their outlets. Here, dam outlet and spillway terminal structure technology is appropriate (USBR 1987). Type II, III, or IV stilling basins, submerged bucket with plunge basin energy dissipators and slotted-grating dissipators can be considered when appropriate to the site conditions. For instance, a plunge basin may have applicability where discharge is to a wet detention pond or a lake.

5.2.1 Impact Stilling Basin

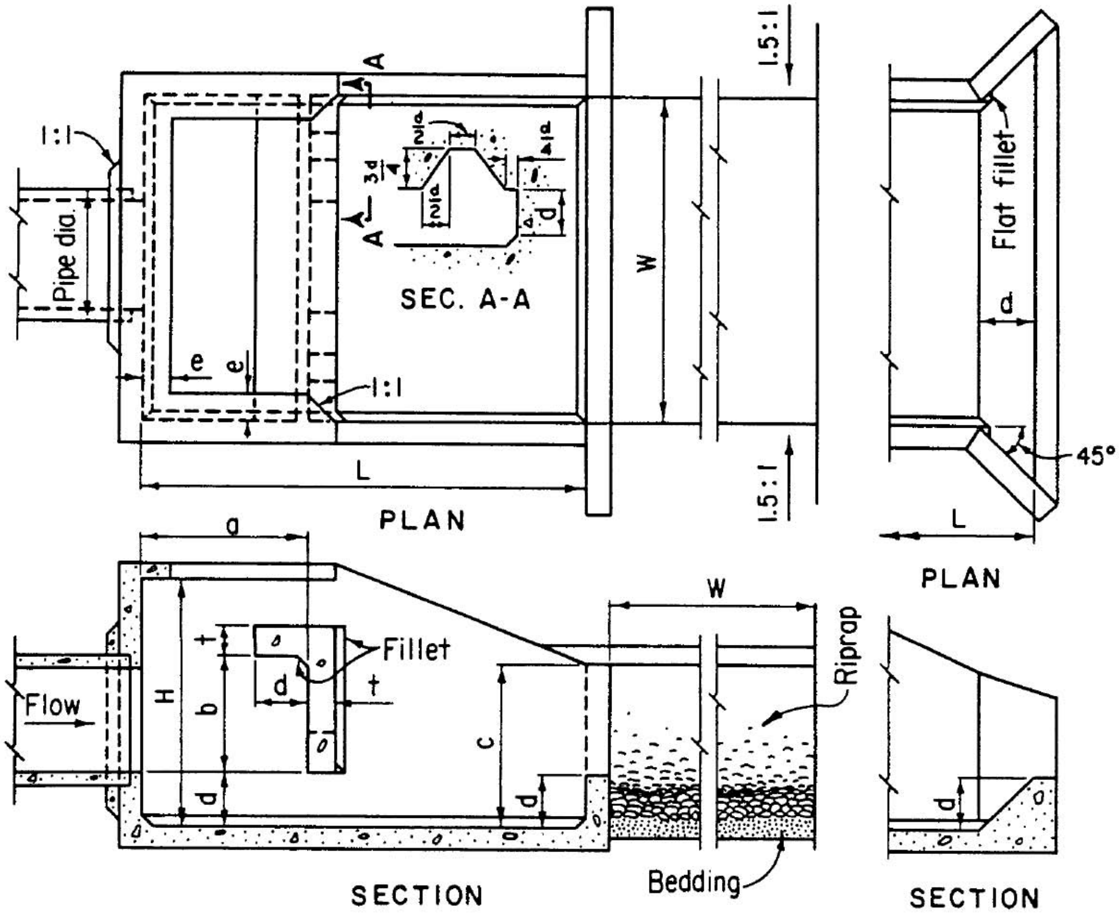
Design standards for an impact stilling basin are based on the USBR Type VI basin, commonly referred to as an impact dissipator or conduit outlet stilling basin. The Type VI basin is a relatively small structure that produces highly efficient energy dissipation characteristics without tailwater control. The original hydraulic design reference by Biechley (1971) presents further model research and includes modifications to the hydraulic design. Additional structural details are provided in Aisenbrey, et al. (1974) and Peterka (1984).

The Type VI basin is designed to operate continuously at the design flow rate. The use of this outlet basin is limited only by structural and economic considerations.

Energy dissipation is accomplished through the turbulence created by the loss of momentum as flow entering the basin impacts a large overhanging baffle. At high flow, further dissipation is produced as water builds up behind the baffle to form a highly turbulent backwater zone. Flow is then redirected under the baffle to the open basin and out to the receiving channel. A check at the basin end reduces exit velocities by breaking up the flow across the basin floor and improves the stilling action at low to moderate flow rates.

The generalized design configuration shown in Figure 19 (Figure 20 for pipe ≤ 36 in.) consists of an open concrete box attached directly to the conduit outlet. The width, W, is determined according to Figure 21 as a function of the Froude number. The sidewalls are high enough to contain most of the splashing during high flows and slope down to form a transition to the receiving channel. The inlet pipe is vertically aligned with an overhanging L-shaped baffle such that the pipe invert is not lower than the bottom of the baffle. The end check height is equal to the height under the baffle to produce tailwater in the basin. The alternate end transition (at 45 degrees) is recommended for grass-lined channels to reduce the overall scour potential just downstream of the check.

The standard USBR design has been modified herein for urban applications to allow drainage of the basin bottom during dry periods. The impact basin can also be adapted to multiple pipe installations. Such modifications are discussed; however, it should be noted that modifications to the design may affect the hydraulic performance of the structure. Model testing is advised for significant changes to the design.



ALTERNATE
END SILL AND
WING WALL

H = 3W/4	c = W/2	
L = 4W/3	d = W/6	
a = W/2	e = W/12	
b = 3W/8	t = W/12	suggested minimum thickness

Figure 19—General Design Dimensions Impact Stilling Basin (USBR VI)

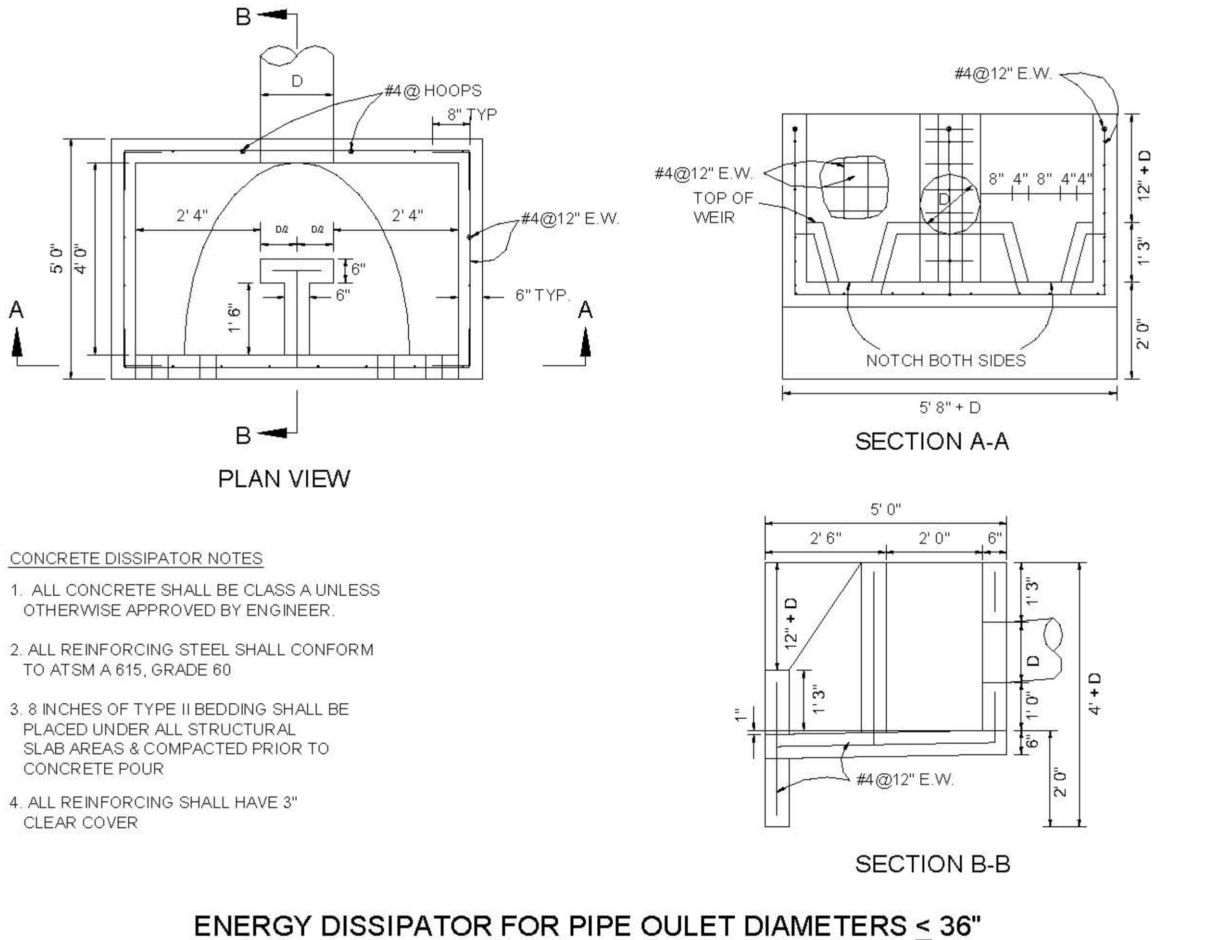
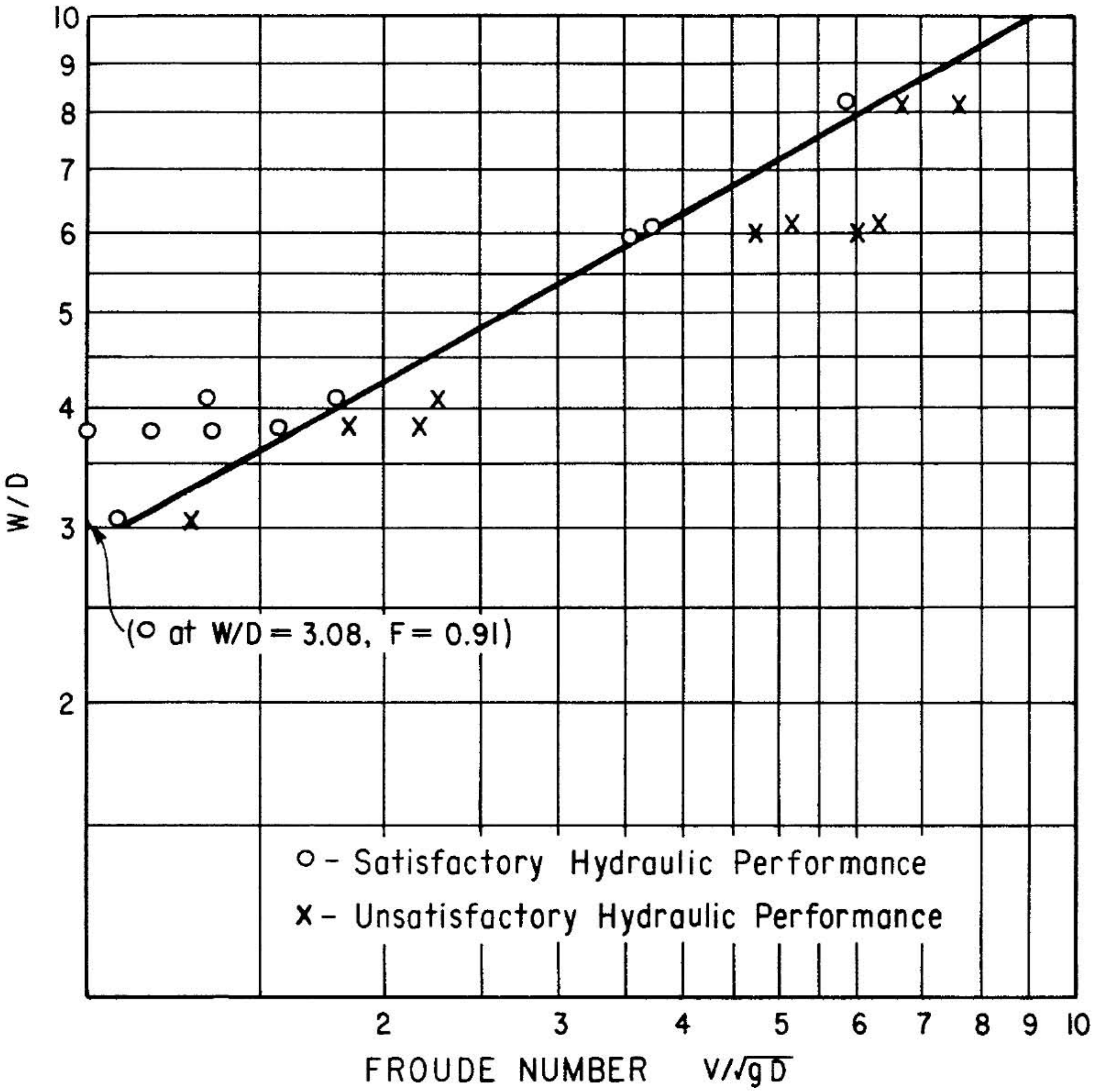


Figure 20—Modifications of Impact Stilling Basin to Allow Basin Drainage for Urban Applications



"W" is the inside width of the basin.
"D" represents the depth of flow entering the basin and is the square root of the flow area at the conduit outlet.
"V" is the velocity of the incoming flow.
The tailwater depth is uncontrolled.

Figure 21—Basin Width Diagram for an Impact Stilling Basin (USBR VI)

5.2.1.1 Low-flow Modifications

The standard design will retain a standing pool of water in the basin bottom that is generally undesirable from an environmental and maintenance standpoint. This situation should be alleviated where practical by matching the receiving channel low-flow invert to the basin invert (see Figure 20).

A low-flow gap is extended through the basin end check wall. The gap in the check should be as narrow as possible to minimize effects on the check hydraulics. This implies that a narrow and deeper (1½- to 2-foot) low-flow channel will work better than a wider gap section. The low-flow width should not exceed 60% of the pipe diameter to prevent the jet from short-circuiting through the cleanout notches.

Low-flow modifications have not been fully tested to date. Caution is advised to avoid compromising the overall hydraulic performance of the structure. Other ideas are possible including locating the low-flow gap at one side (off center) to prevent a high velocity jet from flowing from the pipe straight down the low-flow channel. The optimal configuration results in continuous drainage of the basin area and helps to reduce the amount of siltation.

5.2.1.2 Multiple Conduit Installations

Where two or more conduits of different sizes outlet in proximity, a composite structure can be constructed to eliminate common walls. This can be somewhat awkward since each basin “cell” must be designed as an individual basin with different height, width, etc. Where possible, a more economical approach is to combine storm sewers underground, at a manhole or vault, and bring a single, combined pipe to the outlet structure.

For two conduits of the same size, the outfalls may be combined into a single basin similar to Figure 20. If the design width for each pipe is W, the combined basin width for two pipes would be $1.5W$. Where the flow is different for the two conduits, the design width is based on the higher flow.

The effect of mixing and turbulence of the combined flows in the basin has not been thoroughly model tested to date. It is suggested that no wall be constructed to separate flow behind the baffle, thereby allowing greater turbulence in the combined basin.

Remaining structure dimensions are based on the design width of a separate basin W. If the two pipes have different flow, the combined structure is based on the higher Froude number. Use of a handrail is suggested around the open basin areas where safety is a concern. Access control screens or grating where necessary are a separate design consideration.

5.2.1.3 General Design Procedure

1. Determine the design hydraulic cross-sectional area just inside the pipe, at the outlet. Determine the effective flow velocity, V, at the same location in the pipe. Assume depth and compute the Froude number = $D = \left(A_{sect}\right)^{1/2}$ ar $\dfrac{V}{\left(gD\right)^{1/2}}$

2. The entrance pipe should be turned horizontally at least one pipe diameter equivalent length upstream from the outlet. For pipe slopes greater than 15 degrees, the horizontal length should be a minimum of two pipe diameters.

3. Determine the basin width, W, by entering the appropriate Froude number and effective flow depth into Figure 21. The remaining dimensions are proportional to the basin width according to Figure 19. The basin width should not be oversized since the basin is inherently oversized for less than design flows. Larger basins become less effective as the inflow can pass under the baffle.

4. Structure wall thickness, steel reinforcement, and anchor walls (underneath the floor) should be designed using accepted structural engineering methods. Note that the baffle thickness, t_b, is a suggested minimum. It is not a hydraulic parameter and is not a substitute for structural analysis. Hydraulic forces on the overhanging baffle may be approximated by determination of the hydraulic jet force at the outlet:

   $F_j = 1.94\ V_{out} Q_{des}$ = force in pounds

   Q_{des} = maximum design discharge (cfs)

   V_{out} = velocity of the outlet jet (ft/sec)

5. Type "M" rock riprap should be provided in the receiving channel from the end check to a minimum distance equal to the basin width. The depth of rock should be equal to the check height or at least 2.0 feet. Rock may be buried to finished grades and planted as desired.

6. The alternate end check and wingwall shown in Figure 19 are recommended for all grass-lined channel applications to reduce the scour potential below the check wall.

7. Ideally, the low-flow invert matches the floor invert at the basin end and the main channel elevation is equal to the top of the check. For large basins where the check height, d, becomes greater than the low-flow depth, dimension d in Figure 19 may be reduced by no more than one-third. It should not be reduced to less than 2 feet. This implies that a deeper low-flow channel

(1.5 to 2.0 feet) will be advantageous for these installations. The alternate when *d* exceeds the trickle flow depth is that the basin area will not drain completely.

8. A check section should be constructed directly in front of the low-flow notch to break up bottom flow velocities. The length of this check section should overlap the width of the low flow by about 1 foot. The general layout for the check modifications is shown in Figure 20.

5.2.2 Pipe Outlet to Baffle Chute

The baffle chute developed by the USBR (1958) has also been adapted to use at pipe outlets. This structure is particularly well suited to situations with large conduit outfalls and at outfalls to channels in which some future degradation is anticipated. As mentioned previously, the apron can be extended at a later time to account for channel degradation. Generally, this type of structure is only cost effective if a grade drop is necessary below the outfall elevation.

Figure 22 illustrates a general configuration for a baffled outlet application for a double box culvert outlet. In this case, an expansion zone occurs just upstream of the approach depression. The depression depth is designed as required to reduce the flow velocity at the chute entrance. The remaining hydraulic design is the same as for a standard baffle chute using conditions at the crest to establish the design. The same crest modifications are applicable to allow drainage of the approach depression, to reduce the upstream backwater effects of the baffles, and to reduce the problems of debris accumulation at the upstream row of baffles.

Flow entering the chute should be well distributed laterally across the width of the chute. The velocity should be below critical velocity at the crest of the chute. To insure low velocities at the upstream end of the chute, it may be necessary to provide a short energy dissipating pool. The sequent or conjugate depth in the approach basin should be maintained to prevent jump sweep-out, but the basin length may be considerably less than a conventional hydraulic jump basin since the primary purpose of this pool is only to reduce the average entrance velocity. A basin length of twice the sequent depth will usually provide ample basin length. The end check of the pool may be used as the crest of the chute as shown in Figure 22.

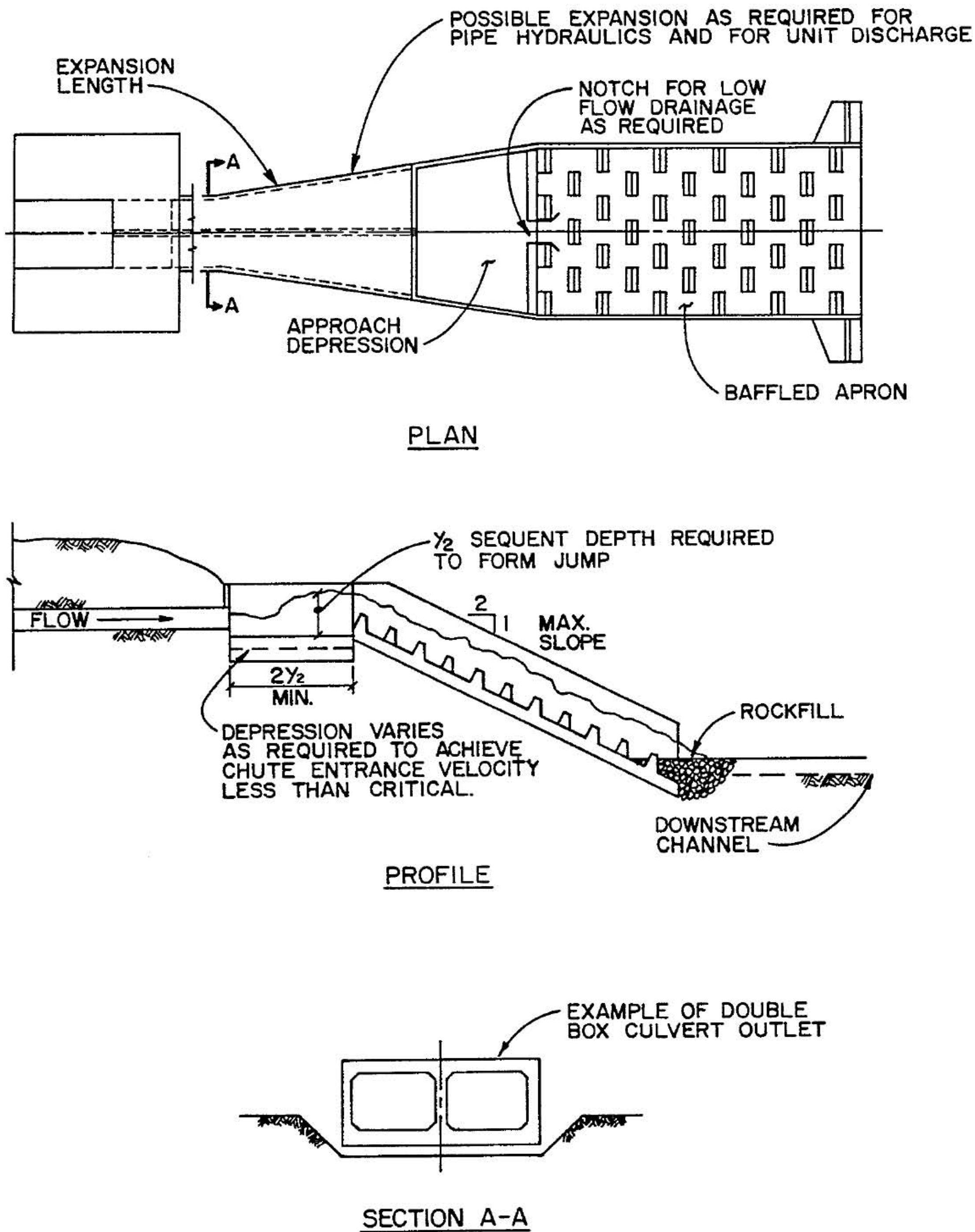
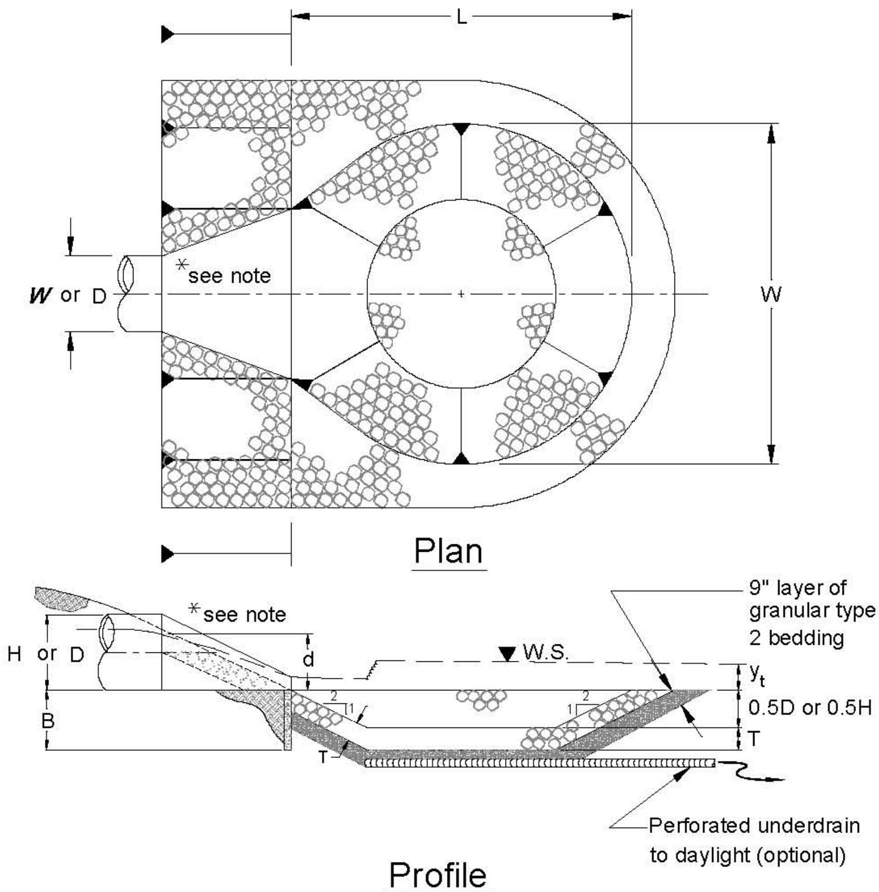


Figure 22—Baffle Chute Pipe Outlet

5.2.3 Low Tailwater Basins

5.2.3.1 General

The design of low tailwater riprap basins for storm sewer pipe outlets and some culverts is necessary when the receiving channel may have little or no flow and uncontrolled pipe velocities would create erosional problems in the channel. Design criteria are provided in Figures 23 through 26.



* Note: For rectangular conduits use a standard design for a headwall with wingwalls, paved bottom between the wingwalls, with an end cutoff wall extending to a minimum depth equal to B

Figure 23—Low Tailwater Riprap Basins for Storm Sewer Pipe Outlets— Low Tailwater Basin at Pipe Outlets (Stevens and Urbonas 1996)

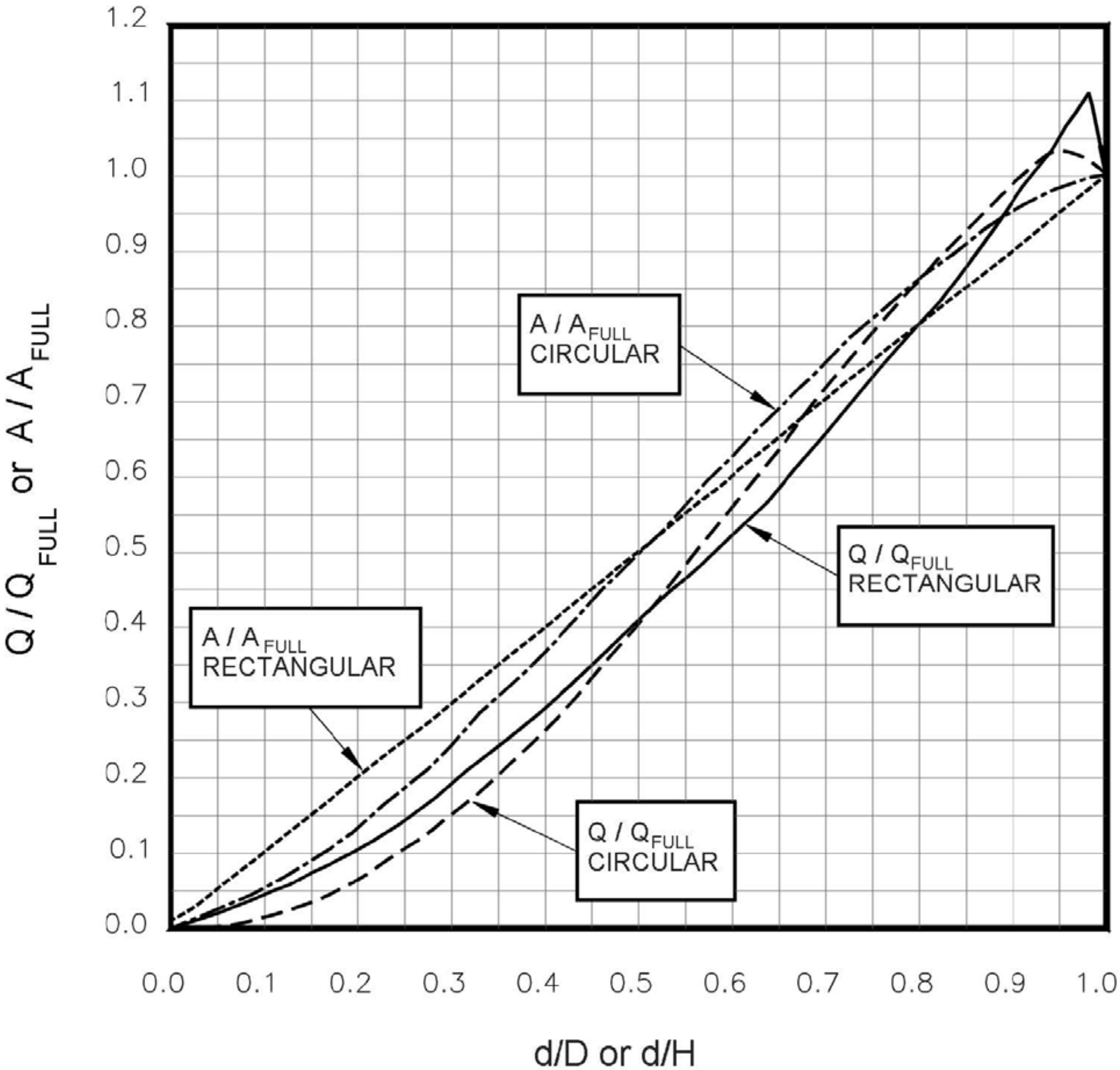


Figure 24—Low Tailwater Riprap Basins for Storm Sewer Pipe Outlets— Discharge and Flow Area Relationships for Circular and Rectangular Pipes (Ratios for Flow Based on Manning's *n* Varying With Depth) (Stevens and Urbonas 1996)

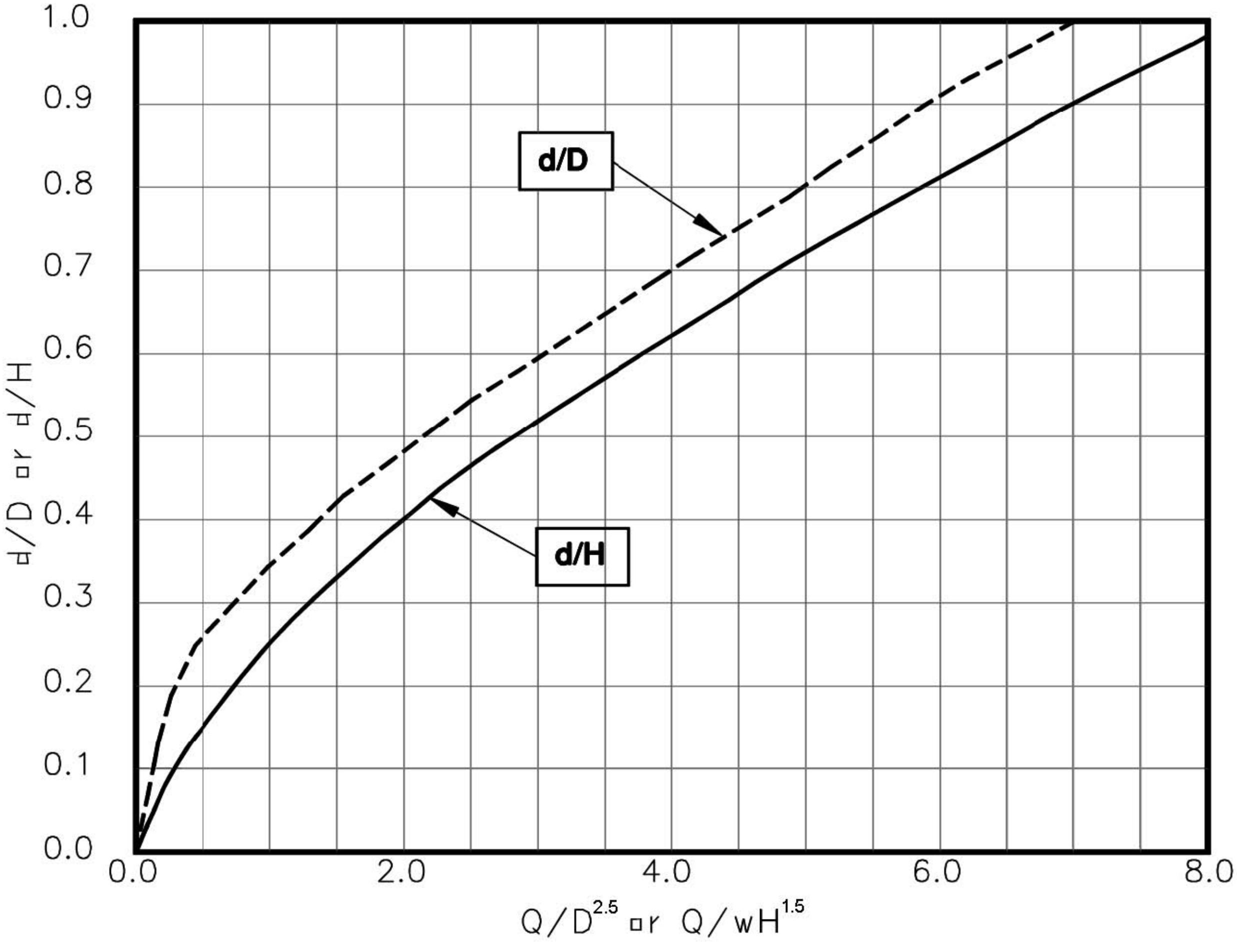


Figure 25—Low Tailwater Riprap Basins for Storm Sewer Pipe Outlets— Brink Depth for Horizontal Pipe Outlets (Stevens and Urbonas 1996)

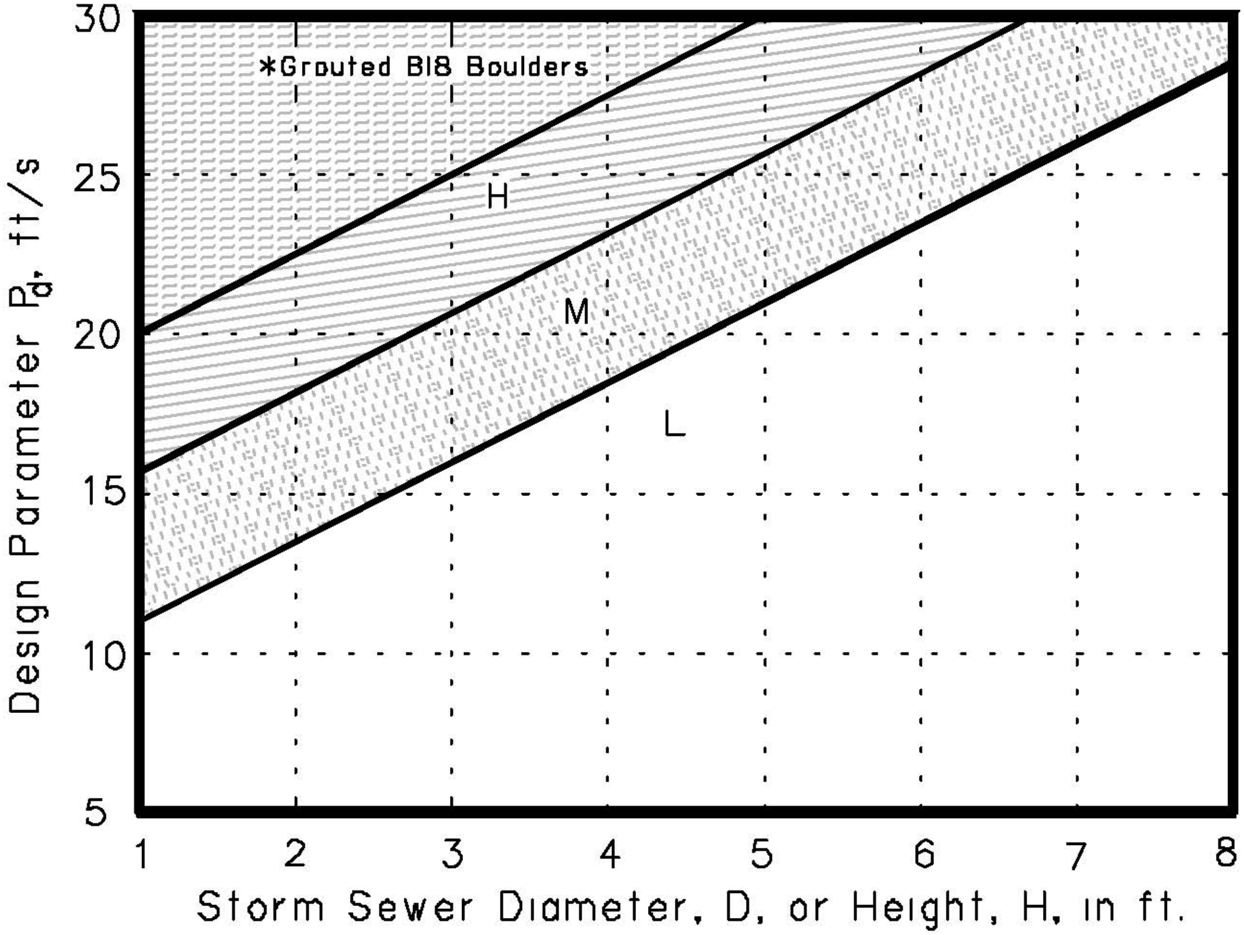


Figure 26—Low Tailwater Riprap Basins for Storm Sewer Pipe Outlets— Riprap Selection Chart for Low Tailwater Basin at Pipe Outlet (Stevens and Urbonas 1996)

5.2.3.2 Objective

By providing a low tailwater basin at the end of a storm sewer conduit or culvert, the kinetic energy of the discharge is dissipated under controlled conditions without causing scour at the channel bottom. Photograph 6 shows a representative low tailwater basin.

Photograph 6—Upstream and downstream views of a low tailwater basin protecting downstream wetland area. Burying and revegetation of the rock would blend the structure better with the adjacent terrain.

5.2.3.3 Low Tailwater Basin Design

For storm sewers, low tailwater is defined as being equal to or less than ⅓ of the storm sewer height, that is:

$$y_t \le \frac{D}{3} \quad \text{or} \quad y_t \le \frac{H}{3}$$

Equation 15

in which:

y_t = tailwater depth at design

D = diameter of circular pipe (ft)

H = height of rectangular pipe (ft)

5.2.3.3.1 Finding Flow Depth and Velocity of Storm Sewer Pipe Outlets

The first step in the design of a scour protection basin at the outlet of a storm sewer is to find the depth and velocity of flow at the outlet. Pipe-full flow can be found using Manning's equation and the pipe-full velocity can be found using the well-known and standard continuity equation. Namely,

$$Q_{full} = \frac{1.49}{n} A_{full} \left(R_{full}\right)^{2/3} S_o^{1/2}$$

Equation 16

in which:

Q_{full} = pipe full discharge at its slope (cfs)

n = Manning's n for the pipe full depth

A_{full} = cross-sectional area of the pipe (ft^2)

S_o = longitudinal slope of the pipe (ft/ft)

R = hydraulic radius of the pipe flowing full, ft [R_{full} = D/4 for circular pipes, $R_{full} = A_{full}/(2H + 2w)$ for rectangular pipes, where D = diameter of a circular conduit, H = height of a rectangular conduit, and w = width of a rectangular conduit (ft)]

Then,

$$V_{full} = Q_{full} / A_{full}$$

Equation 17

In which:

V_{full} = flow velocity of the pipe flowing full (ft/sec)

The normal depth of flow, d, and the velocity at that depth in a conduit can be found with the aid of Figure 24. Using the known design discharge, Q, and the calculated pipe-full discharge, Q_{full}, enter Figure 24 with the value of Q/Q_{full} and find d/D for a circular pipe of d/H for a rectangular pipe.

Compare the value of this d/D (or d/H) with that obtained from Figure 25 using the Froude parameter, namely,

$$Q/D^{2.5} \quad \text{or} \quad Q/\left(wH^{1/5}\right)$$

Equation 18

Choose the smaller of the two (d/D or d/H) ratios to calculate the flow depth at the end of the pipe, namely,

$$d = D\left(d/D \quad \text{or} \quad d = H\left(d/H\right)$$

Equation 19

Again, enter Figure 24 using the smaller *d/D* (or *d/H*) ratio to find the A/A_{full} ratio. Use this to calculate the area of flow at the end of the pipe, namely,

$$A = \left(A/A_{full}\right)A_{full}$$

Equation 20

in which:

A = area of the design flow in the end of the pipe (ft^2)

Finally,

$$V = Q/A$$

Equation 21

in which:

V = design flow velocity at the pipe outlet (ft/sec)

5.2.3.3.2 Riprap Size

For the design velocity, use Equation 22 to find the size and type of the riprap to use in the scour protection basin downstream of the pipe outlet (i.e., B18, H, M or L). First, calculate the riprap sizing design parameter, P_d , namely,

$$P_d = \left(V^2 + gd\right)^{1/2}$$

(Equation 22)

in which:

V = design flow velocity at pipe outlet (ft/sec)

g = acceleration due to gravity = 32.2 ft/sec^2

d = design depth of flow at pipe outlet (ft)

When the riprap sizing design parameter indicates conditions that place the design above the Type H riprap line in Figure 26, use B18, or larger, grouted boulders. An alternative to a grouted boulder or loose riprap basin is to use the standard USBR Basin VI, as described in Section 3.2.

After the riprap size has been selected, the minimum thickness of the riprap layer, T, in feet, in the basin is set at:

$$T = 1.75D_{50}$$

(Equation 23)

in which:

D_{50} = the median size of the riprap (see Table 2.)

Table 2
Median (i.e., D50) Size of Riprap

Riprap Type	D_{50}—Median Rock Size (inches)
L	9
M	12
H	18
B18	18 (grouted)

5.2.3.3.3 Basin Length

The minimum length of the basin, *L*, in Figure 23, is defined as being the greater of the following lengths:

for circular pipe,

$$L = 4D \quad \text{or} \quad L = (D)^{1/2}\left(\frac{V}{2}\right)$$

(Equation 24)

for rectangular pipe,

$$L = 4H \quad \text{or} \quad L = (H)^{1/2}\left(\frac{V}{2}\right)$$

(Equation 25)

in which:

L = basin length (Figure 23)

H = height of rectangular conduit

V = design flow velocity at outlet

D = diameter of circular conduit

5.2.3.3.4 Basin Width

The minimum width, *W*, of the basin downstream of the pipe's flared end section is set as follows:

for circular pipes,

$$W = 4D$$

(Equation 26)

for rectangular pipe,

$$W = w + 4H$$

(Equation 27)

in which:

W = basin width (Figure 23)

D = diameter of circular conduit

w = width of rectangular conduit

5.2.3.3.5 Other Design Requirements

All slopes in the pre-shaped riprapped basin are 2H to 1V.

Provide pipe joint fasteners and a structural concrete cutoff wall at the end of the flared end section for a circular pipe or a headwall with wingwalls and a paved bottom between the walls, both with a cutoff wall that extends down to a depth of

$$B = \frac{D}{2} + T \quad \text{or} \quad B = \frac{H}{2} + T$$

(Equation 28)

in which:

B = cutoff wall depth

D = diameter of circular conduit

T = Equation 23

The riprap must be extended up the outlet embankment's slope to the mid-pipe level.

6.0 DESIGN EXAMPLE

The following example problem illustrates the culvert design procedures using the FHWA nomographs and using Culvert Spreadsheet application.

6.1 Culvert Under an Embankment

Given: $Q_{5\text{-yr}}$ = 20 cfs, $Q_{100\text{-yr}}$ = 35 cfs, L = 95 feet

The maximum allowable headwater elevation is 5288.5. The natural channel invert elevations are 5283.5 at the inlet and 5281.5 at the outlet. The tailwater depth is computed as 2.5 feet for the 5-year storm, and 3.0 feet for the 100-year storm.

Solution:

Step 1. Fill in basic data (Figure 27)

$Q_{5\text{-yr}}$ = discharge for 5-year storm

$Q_{100\text{-yr}}$ = discharge for 100-year storm

Headwater and tailwater elevations

Step 2. Set invert elevations at natural channel invert elevations to avoid scour. Compute s and $L/(100s)$.

Step 3. Start with an assumed culvert size for the 5-year storm by adopting a velocity of 6.5 ft/sec. In this case, first size is estimated by adopting a velocity of 6.5 ft/sec and computing A = 20/6.5 = 3.1 ft², giving a culvert diameter, D = 24 inches.

Step 4. For this example, two inlets are considered: square edge with headwall (K_e = 0.4) and groove end with headwall (K_e = 0.2). Also, assume concrete pipe will be used with a Manning's n of 0.012 (Note: the District recommends a minimum n of 0.013; however, 0.012 is used in this example to correspond to the FHWA nomograph.)

Step 5. Using the inlet control nomograph (Figure 28), the ratio of the headwater depth to the culvert diameter (Hw/D) is 1.47 for the square edge and 1.32 for the groove end. Thus, the inlet control headwater depths are 2.94 feet and 2.64 feet, respectively.

Step 6. The outlet control headwater depth is determined using the method described in Section 3.0. The head is determined from the nomograph (Figure 29). The resulting outlet control headwater depths are 2.13 feet for the square edge and 1.90 feet for the grove end inlet.

Step 7. Comparing the headwater depths for inlet control (2.94 feet and 2.64 feet) and outlet control (2.13 feet and 1.90 feet) shows that the culvert is inlet controlled with either inlet configuration. Furthermore, the calculated headwater depths are less than the allowable headwater depth. These results can also be determined using the UD-Culvert Spreadsheet.

Step 8. The next step is to evaluate the culvert for the 100-year flow of 35 cfs and tailwater depth of 3.0 feet. Using the same procedure, the culvert continues to be inlet controlled with the square-edge inlet and switches to outlet control with the more efficient groove-end inlet. However, both of the calculated headwater depths exceed the allowable headwater depth and, consequently, are not viable alternatives.

Step 9. Increase the pipe diameter to 27 inches and repeat the process. The resulting headwater depths are less than the allowable.

Step 10. Compute outlet velocities for each acceptable alternate.

Step 11. Compute cost for each alternate.

Step 12. Make recommendations.

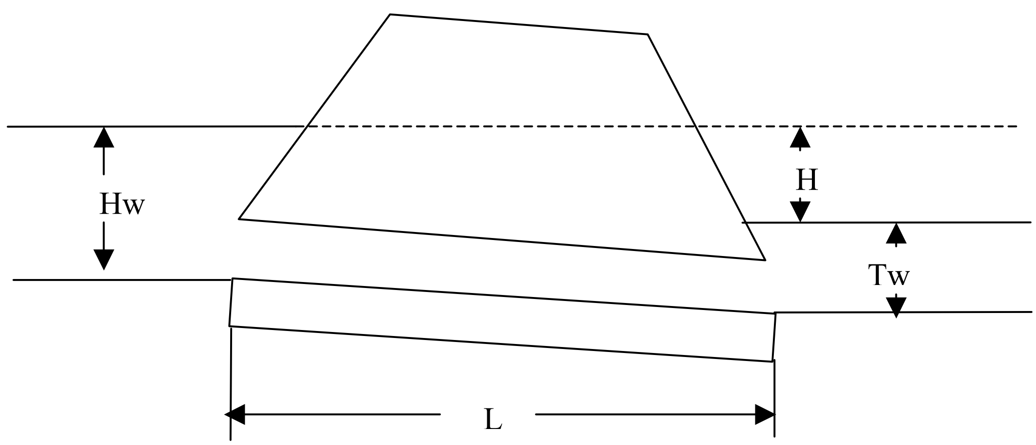
6.2 Culvert Design Example[1]

A culvert will be needed to convey surface waters under an embankment that will be constructed in a new residential development near Springfield, MO. According to preliminary field surveys, the natural channel invert elevation is 1200 feet at the inlet, 1198 feet at the outlet, and the horizontal length of the culvert under the embankment will be 95 feet. To insure adequate freeboard, to be consistent with city risk mitigation policies, and to satisfy all relevant federal and state standards, the maximum allowable

[1] Disclaimer: The actual design of a culvert installation is more difficult, and involves a larger amount of engineering judgment, than the simple process of sizing a culvert. Problems that may be encountered are too varied and too numerous to create a generalized design process appropriate for every situation. Many other factors are important for the successful design of a safe, cost-effective culvert including: amount and type of cover and vegetation, public safety issues (trash racks, etc.), pipe coatings and sealants, fish passage issues, multipurpose objectives, available construction techniques, cost, and protection against abrasion and erosion – not to mention structural and geotechnical considerations. This example is intended to illustrate some of the basic design steps for a simple inlet controlled, circular concrete pipe embankment culvert that will be constructed without modification of the natural grade. Design engineers are advised to use the automatic design tools and nomographs with caution and

headwater elevation for this culvert site is 1205 feet (five feet above the inlet invert elevation). Runoff calculations for the catchment area yielded peak flows of 20 cfs for the 5-year event and 35 cfs for the 100-year event. Downstream channel inspections yielded tailwater depths for these flows of 2.5 feet and 3.0 feet respectively. Determine the hydraulic design aspects for a circular concrete culvert that will satisfy these constraints through hand calculations, using the FHWA nomographs, and using an automatic spreadsheet application. Investigate various culvert diameters, inlet types (square edge with headwall-K_e=0.4, groove end with headwall-K_e=0.2), and outlet velocities. Discuss the results and provide any additional recommendations.

Determining the size of a culvert is a trial and error design process. Experience suggests that a 24-inch culvert diameter may be able to safely convey these design flows under the embankment. The culvert slope is ***s***=2ft/95ft ~ 0.021 ft/ft. Since the culvert outlet will be submerged for both the 5- and 100-year storm events (given the initial diameter choice), a reasonable initial guess is that both inverts will be submerged and the flow in the culvert will be full for the entire length of the culvert. This is the classic full-flow, outlet-controlled condition with the barrel in pressure flow throughout its length. This condition is often assumed for hand calculations, but according to the Federal Highway Administration (FHWA) document on the hydraulic design of highway culverts, it seldom actually exists.

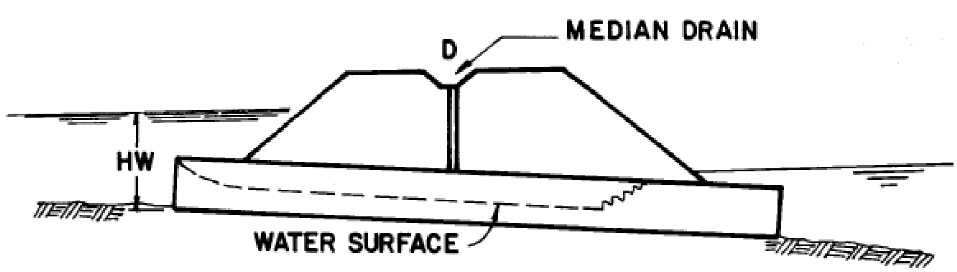


For full flow the difference in the headwater and tailwater elevations, H, is given by: $H = \left[1 + k_e + \frac{K_u n^2 L}{R^{1.33}}\right]\frac{V^2}{2g}$ with K_u=29 (English Units). Using g=32.2ft/s^2, n=0.012 (0.013 is often used for design, but the nomographs provided by the FHWA assume the use of n=0.012),[2] L=95 feet,

to refer to the FHWA Publication No. FHWA-NHI-01-020 on the Hydraulic Design of Highway Culverts for any but the simplest designs.
[2] Nomographs are available for some different Manning's n values. If the desired nomograph is not available the correction procedure described in FHWA-NHI-01-020 may be used.

Q=20cfs, D=2 feet, and K_e=0.2, gives H~1.38 feet and a headwater depth, Hw, of Hw~1.90feet. This is less than the 5 foot maximum headwater, but note that this headwater depth does not correspond to a submerged inlet. We also must check the inlet control headwater depth. If the inlet control headwater depth is higher than the outlet control depth then the culvert may operate in either flow configuration, or oscillate between the two. The concept of "minimum performance" applies so that while the culvert may operate more efficiently at times (more flow for a given headwater level), it will never operate at a lower level of performance than calculated.

For inlet control the governing equations are the weir flow equations and the orifice flow equations with a poorly defined transitional region between the two regimes. The FHWA inlet control nomographs were developed from data compiled from a large amount of research conducted by the National Bureau of Standards (NBS). Using the nomograph for inlet control for the grooved end with headwall inlet yields a headwater control depth of Hw~2.64 feet.[3] Since this is greater than the 1.9 feet calculated for outlet control the culvert will be inlet controlled *even though both ends of the pipe will be submerged.* The flow in the pipe will be supercritical and there will be a jump somewhere within the culvert. This case is illustrated below and reinforces the statement that there is not a simple, universal method to culvert with simple calculations that can be done by hand. This culvert would, in fact, likely require a median inlet to ventilate the culvert barrel. If the barrel were not ventilated, sub-atmospheric pressures could develop which might create an unstable condition during which the barrel would alternate between full flow and partly full flow.



The 24-inch culvert is thus adequate for the 20cfs flows of the 5-year event. However, when the headwater depths are calculated (both for inlet and outlet control) for the 35 cfs flows of the 100-year event the headwater depths exceed the maximum allowable 5 feet. The pipe must be enlarged[4] and the process continues until all the criteria are satisfied. (See the completed Design Computation Form and

[3] The use of the nomographs to determine the headwater values for this example are included.
[4] Other approaches to reducing the headwater include inlet depressions, downstream modifications to reduce the tailwater depth, and tapered inlets, etc.

the included nomographs for the numeric details of these calculations. An example of an automated spreadsheet program output for the design is also included. Note that it uses different variable names.) The exit velocities should be checked and energy dissipation structures should be designed as needed. Cost should help determine which inlet configuration to use in the event that multiple configurations satisfy the design constraints.

PROJECT: Drainage Criteria Manual
Culvert Design Example

DESIGNER: ____________
DATE: ____________

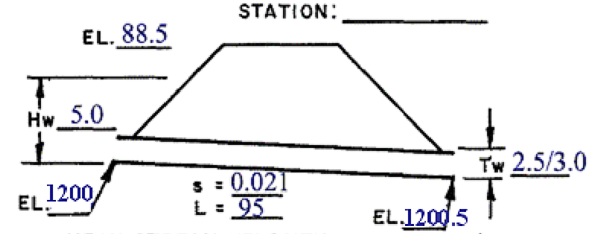
HYDROLOGIC AND CHANNEL INFORMATION

Q_1 = 20 cfs TAILWATER ELEVATION = 1200.5
Q_2 = 35 cfs TAILWATER ELEVATION = 1200

(Q_1 = DESIGN DISCHARGE, SAY Q_{25}
Q_2 = CHECK DISCHARGE, SAY Q_{50} OR Q_{100})

SKETCH

STATION: ____________



MEAN STREAM VELOCITY = ______
MAX. STREAM VELOCITY = ______

$\frac{L}{100s}$ = 45

CULVERT DESCRIPTION w/ Headwall (ENTRANCE TYPE)	Q	SIZE	HEADWATER COMPUTATION: INLET CONT. $\frac{Hw}{D}$	INLET CONT. Hw	OUTLET CONTROL $HW = H + h_o - Ls$: K_e	H	d_c	$\frac{d_c+D}{2}$	Tw	h_o	Ls	Hw	CHART No.	CHART Hw	CONTROLLING HW	OUTLET VELOCITY	COST	COMMENTS
RCP w/ sq edge	20	24	1.47	2.94	0.4	1.63	0.80	1.40	2.5	2.5	2.0	2.13			2.94	6.4		Inlet Control
RCP w/ groove	20	24	1.32	2.64	0.2	1.40	0.80	1.40	2.5	2.5	2.0	1.90			2.64	6.4		Inlet Control
RCP w/ sq edge	35	24	3.08	6.16	0.4	4.90	0.96	1.48	3.0	3.0	2.0	5.90			6.16	11.1		Inlet Control HW>allowable
RCP w/ groove	35	24	2.50	5.00	0.2	4.40	0.96	1.48	3.0	3.0	2.0	5.40			5.40	11.1		Outlet Control HW>allowable
RCP w/ sq edge	35	27	2.03	4.57	0.4	2.85	1.01	1.63	3.0	3.0	2.0	3.85			4.57	8.8		Inlet Control
RCP w/ groove	35	27	1.71	3.85	0.2	2.50	1.01	1.63	3.0	3.0	2.0	3.50			3.85	8.8		Inlet Control

SUMMARY & RECOMENDATIONS: Choose 27-inch RCP to pass the 100-year flow within the allowable headwater.
Either entrance type will work; allow cost to determine.
Design outlet protection based on velocity of 8.8 ft/sec.

From BPR

Figure 27—Design Computation Form for Culverts—Example 9.1

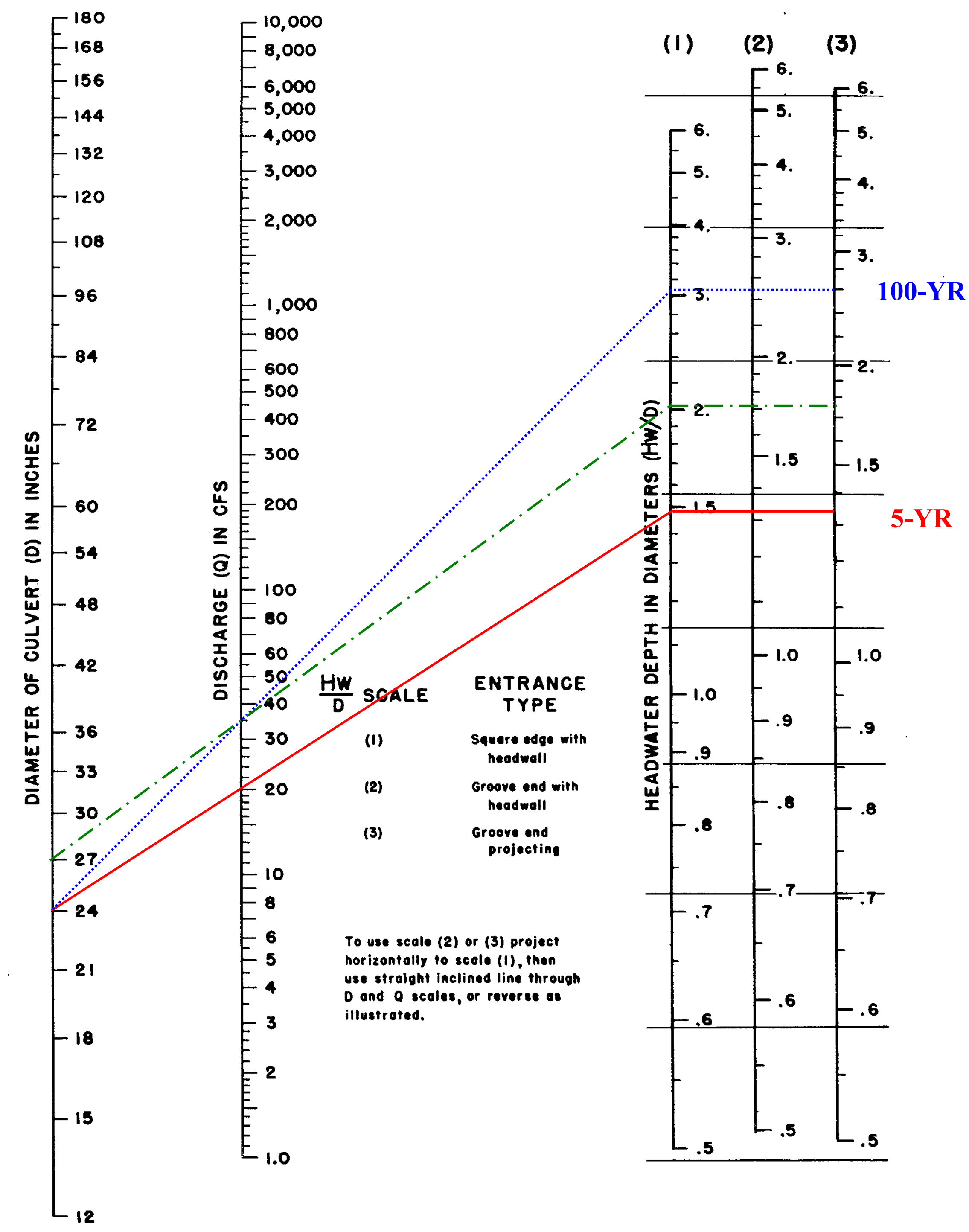
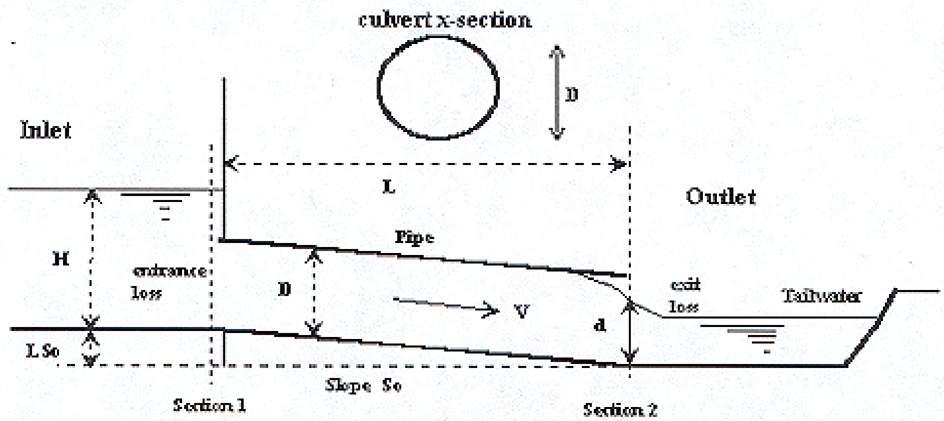


Figure 28—Headwater Depth for Concrete Pipe Culverts with Inlet Control—Example 9.1

Headwater Depth For Circular Culvert

Project: Drainage Criteria Manual
Pipe ID: Example 9.1



Design Information (input)

Design Discharge	Q =	20.0	cfs
Pipe Diameter	D =	2.00	ft
Inlet Edge Type (choose from pull-down list)	Inlet Type =	Square End with Headwall	
Inlet Invert Elevation	I_e =	83.50	ft
Outlet Invert Elevation	O_e =	81.50	ft
Pipe Length	L =	95.0	ft
Manning's Roughness N-Value	N =	0.012	*
Bend Loss Coefficient	K_b =	0.00	
Exit Loss Coefficient	K_x =	0.00	
Tailwater Water Surface Elevation	El. Y_t =	84.00	ft

Calculations (output)

Pipe Cross Sectional Area	A_o =	3.14	sq ft
Culvert Slope	S_o =	0.0211	ft/ft
Normal Flow Depth	Y_n =	0.54	ft
Critical Flow Depth	Y_c =	0.80	ft
Headwater Depth by Inlet Control			
Headwater Depth by Inlet Control	HW-inlet=	2.92	ft
Headwater Depth by Outlet Control			
Tailwater Depth for Design	d =	2.50	ft
Friction Loss Coefficient over Culvert Length	K_f =	1.00	
Sum of All Loss Coefficients	K_s =	1.50	
Headwater Depth by Outlet Control	HW-outlet=	2.07	ft
Design Headwater Depth	HW=	2.92	ft
HW/D Ratio =	HW/D=	1.46	

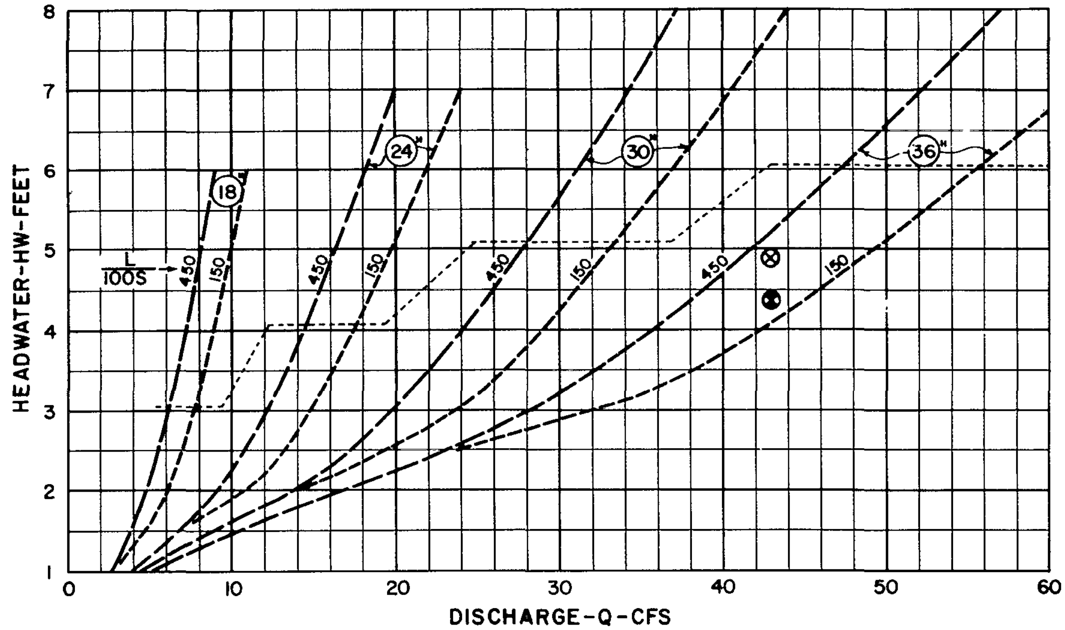
Figure 29—Head for Concrete Pipe Culverts Flowing Full (*n* = 0.012)—Example 9.1

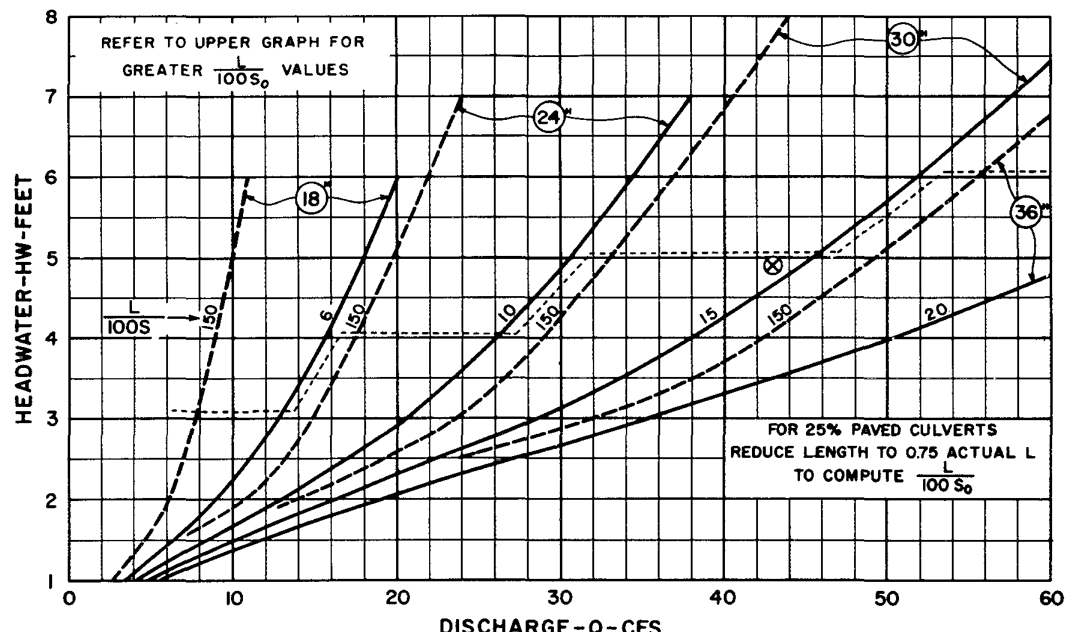
7.0 CHECKLIST

Criterion/Requirement	✓
Culvert diameter should be at least 18 inches.	
Evaluate the effects of the proposed culvert on upstream and downstream water surface elevations.	
When retrofitting or replacing a culvert, evaluate the changes in the upstream and downstream flood hazard.	
Review any proposed changes with local, state, and federal regulators.	
When a culvert is sized such that the overlying roadway overtops during large storms, check the depth of cross flow with Table DP-3 in the POLICY chapter.	
Provide adequate outlet protection in accordance with the energy dissipator discussion in the MAJOR DRAINAGE and HYDRAULIC STRUCTURES chapters.	

8.0 CAPACITY CHARTS AND NOMOGRAPHS

Capacity charts and nomographs covering the range of applications commonly encountered in urban drainage are contained in this section. These charts are from the FHWA Hydraulic Design Series No. 5 (FHWA 1985), which also contains detailed instructions for their use. For situations beyond the range covered by these charts, reference should be made to the original publications.





From BPR
EXAMPLE

⊗ GIVEN:
43 CFS; AHW = 4.9 FT.
L = 72 FT.; S_0 = 0.003

SELECT 36" UNPAVED
HW = 4.4 FT.

Figure 30—Culvert Capacity Standard Circular Corrugated Metal Pipe Headwall Entrance 18" to 36"

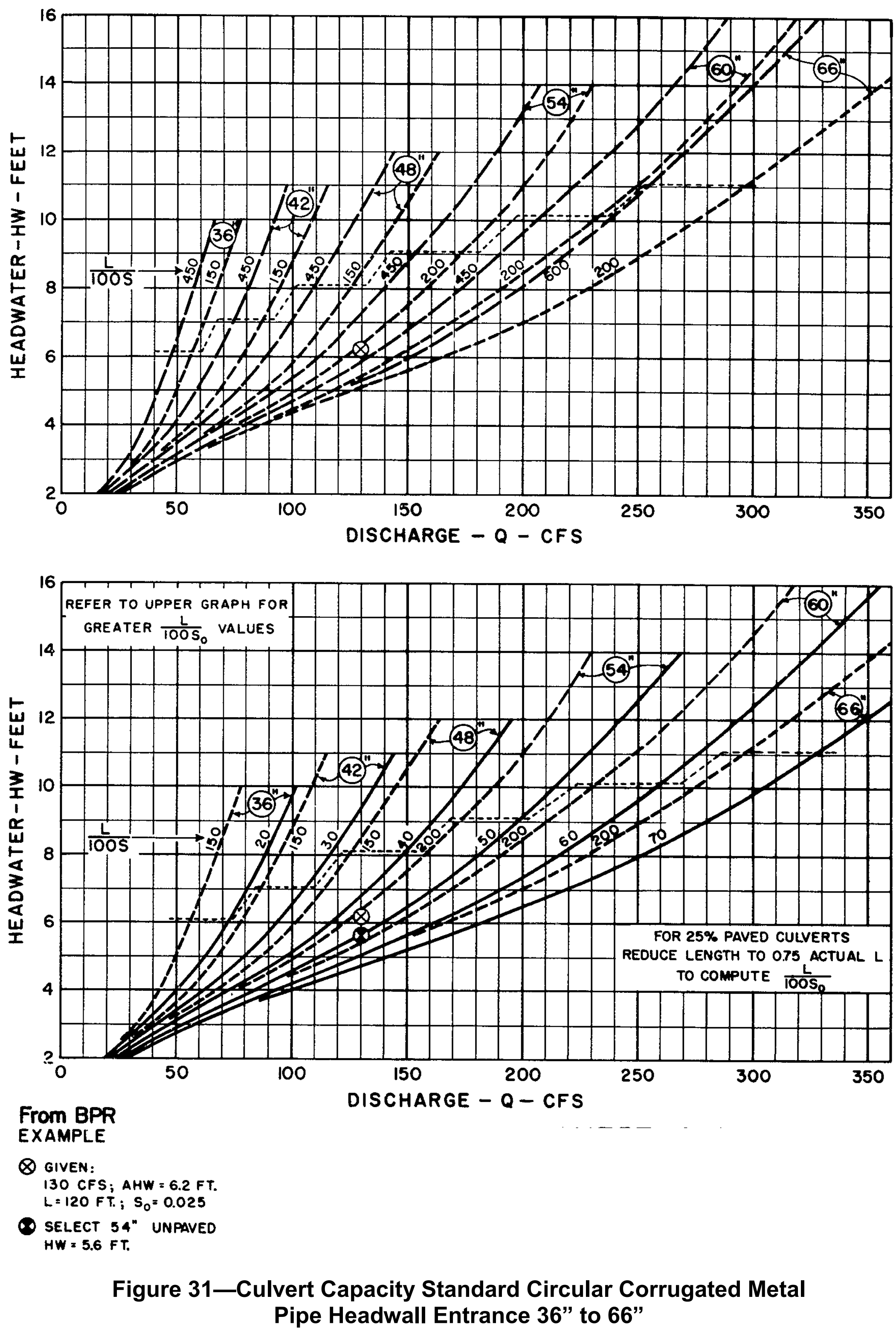
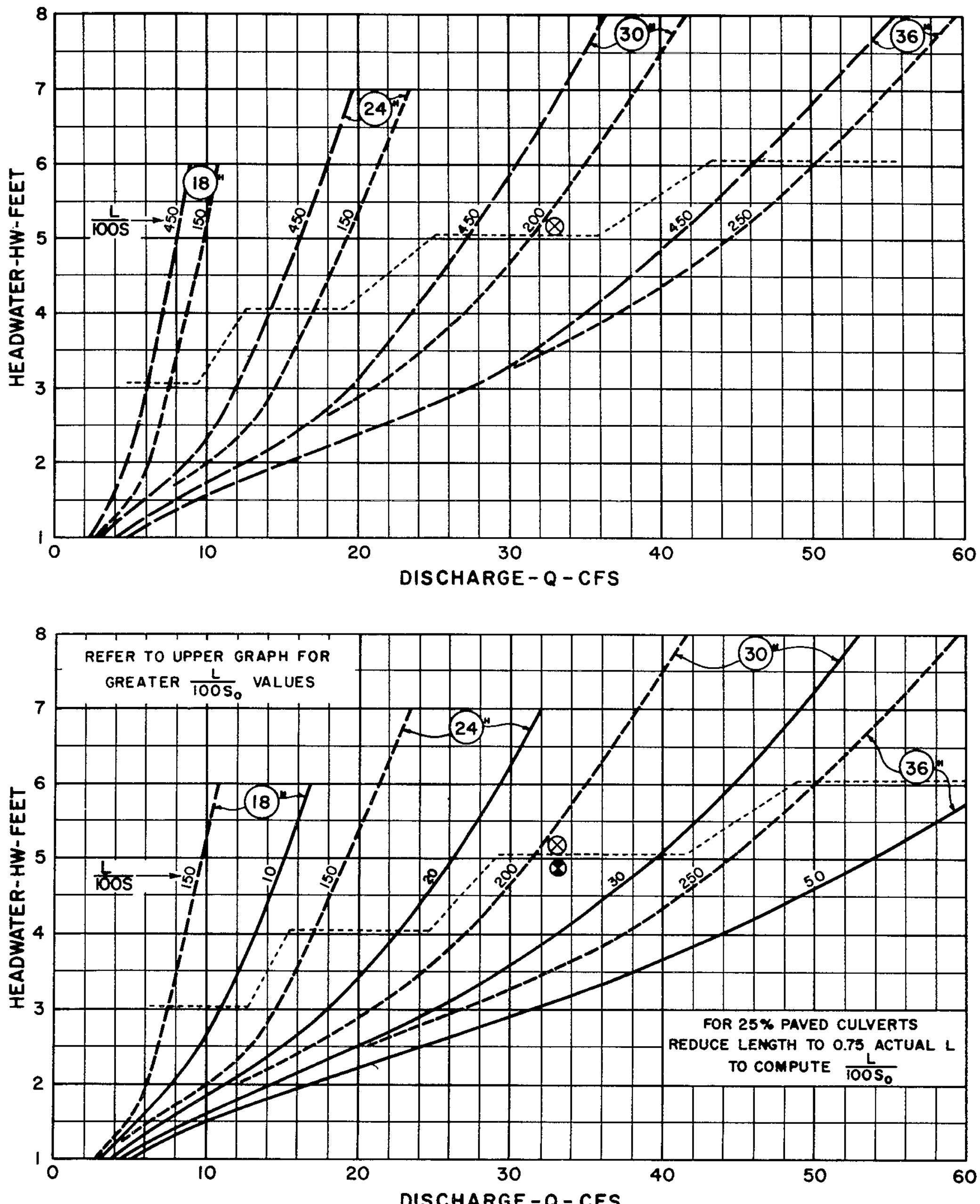


Figure 31—Culvert Capacity Standard Circular Corrugated Metal Pipe Headwall Entrance 36" to 66"



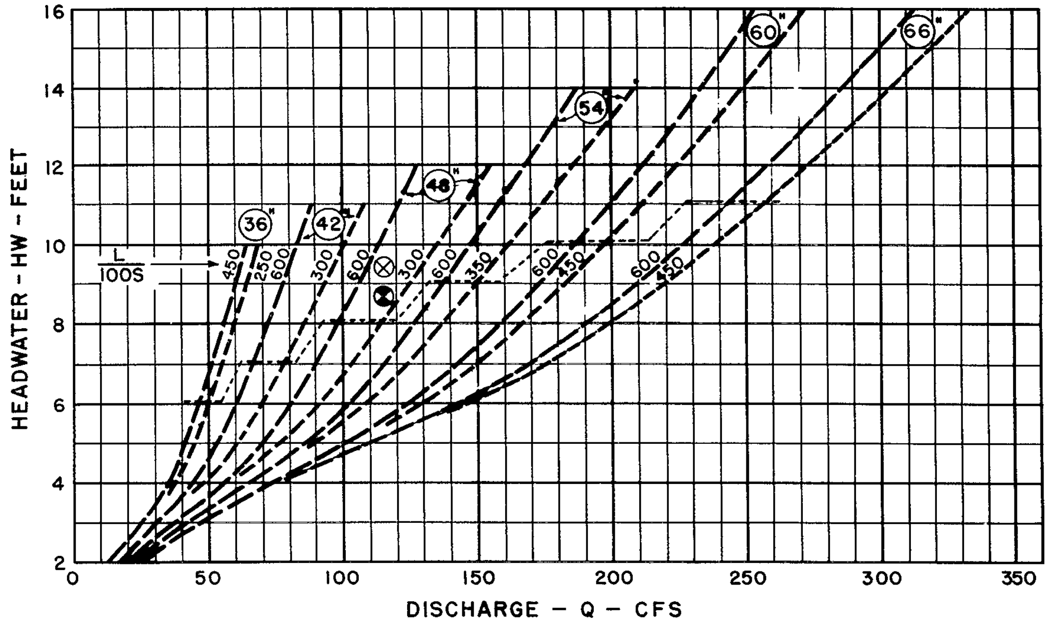
From BPR

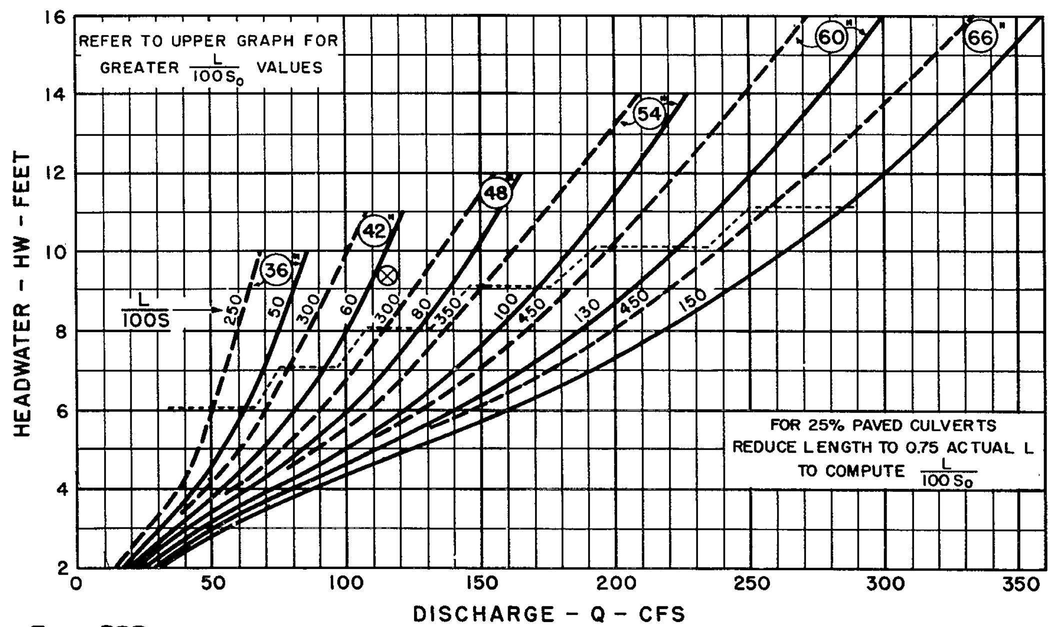
EXAMPLE

⊗ GIVEN:
33 CFS; AHW = 5.2 FT.
L = 70 FT.; S_0 = 0.005

⊗ SELECT 30" UNPAVED
HW = 4.9 FT.

Figure 32—Culvert Capacity Standard Circular Corrugated Metal Pipe Projecting Entrance 18" to 36"





From BPR

EXAMPLE

⊗ GIVEN:
115 CFS; AHW = 9.4 FT.
L = 135 FT.; S_0 = 0.0034

SELECT 48" UNPAVED
HW = 8.6 FT.

Figure 33—Culvert Capacity Standard Circular Corrugated Metal Pipe Projecting Entrance 36” to 66”

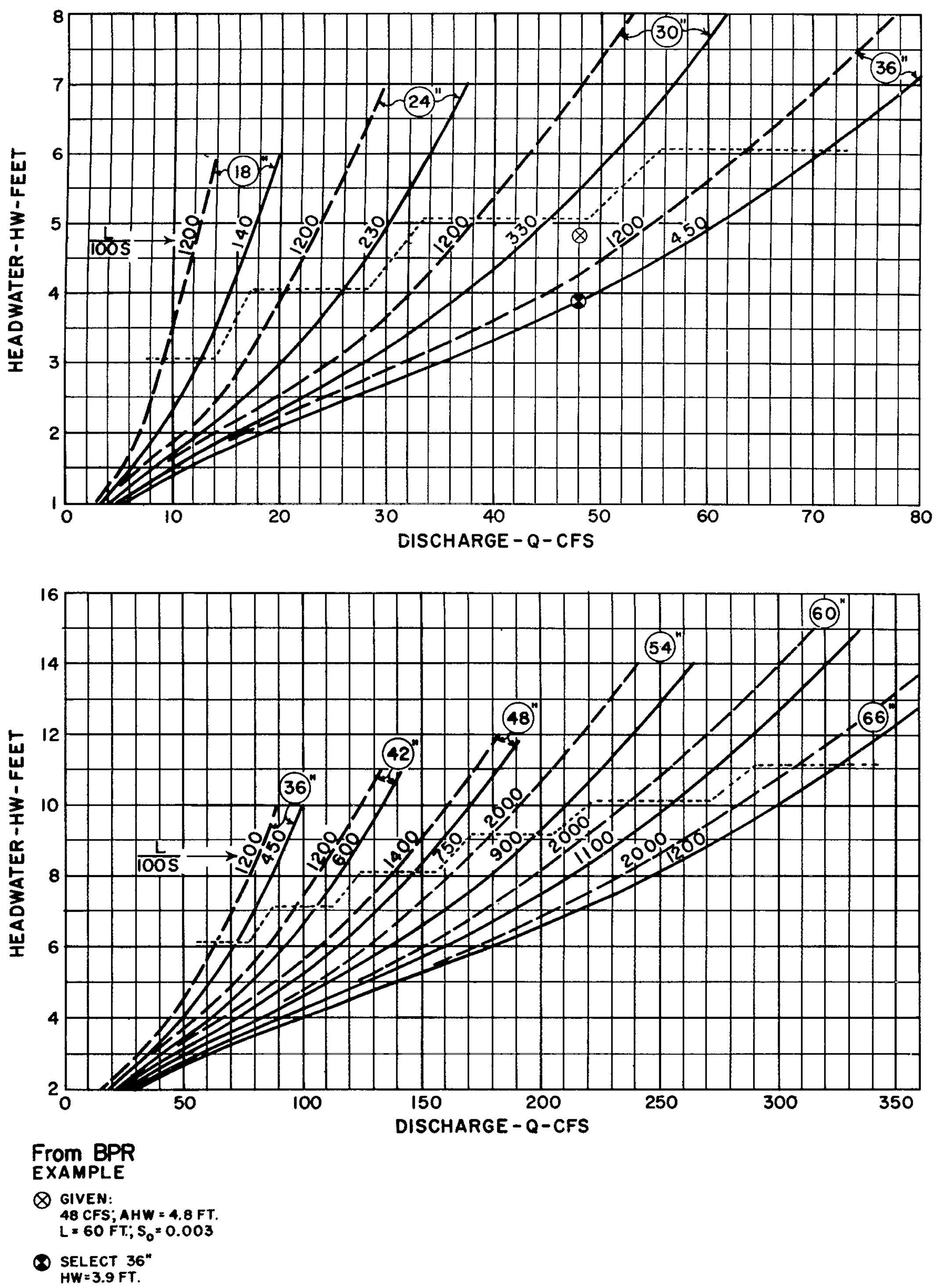


Figure 34—Culvert Capacity Circular Concrete Pipe Square-Edged Entrance 18" to 66"

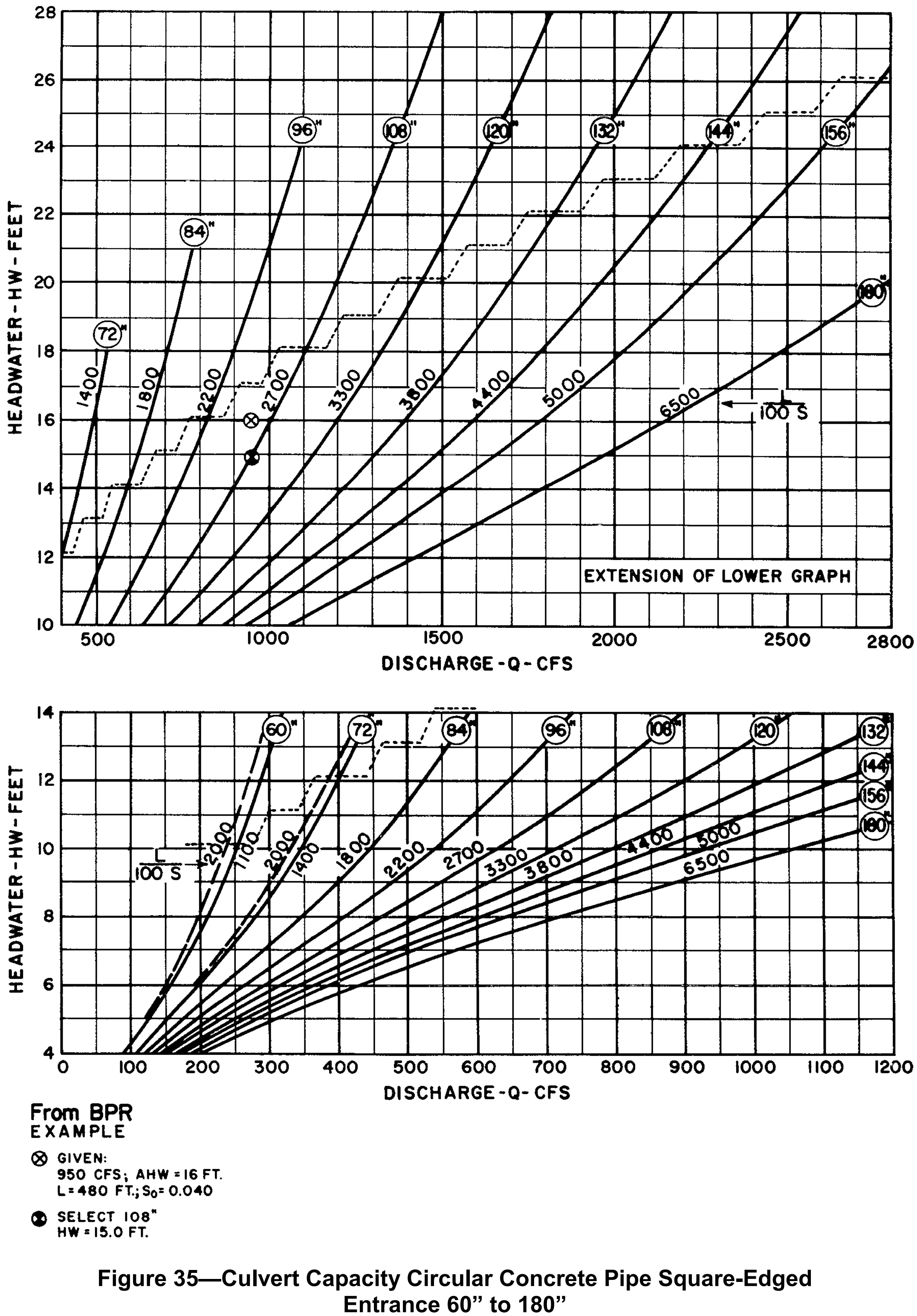


Figure 35—Culvert Capacity Circular Concrete Pipe Square-Edged Entrance 60" to 180"

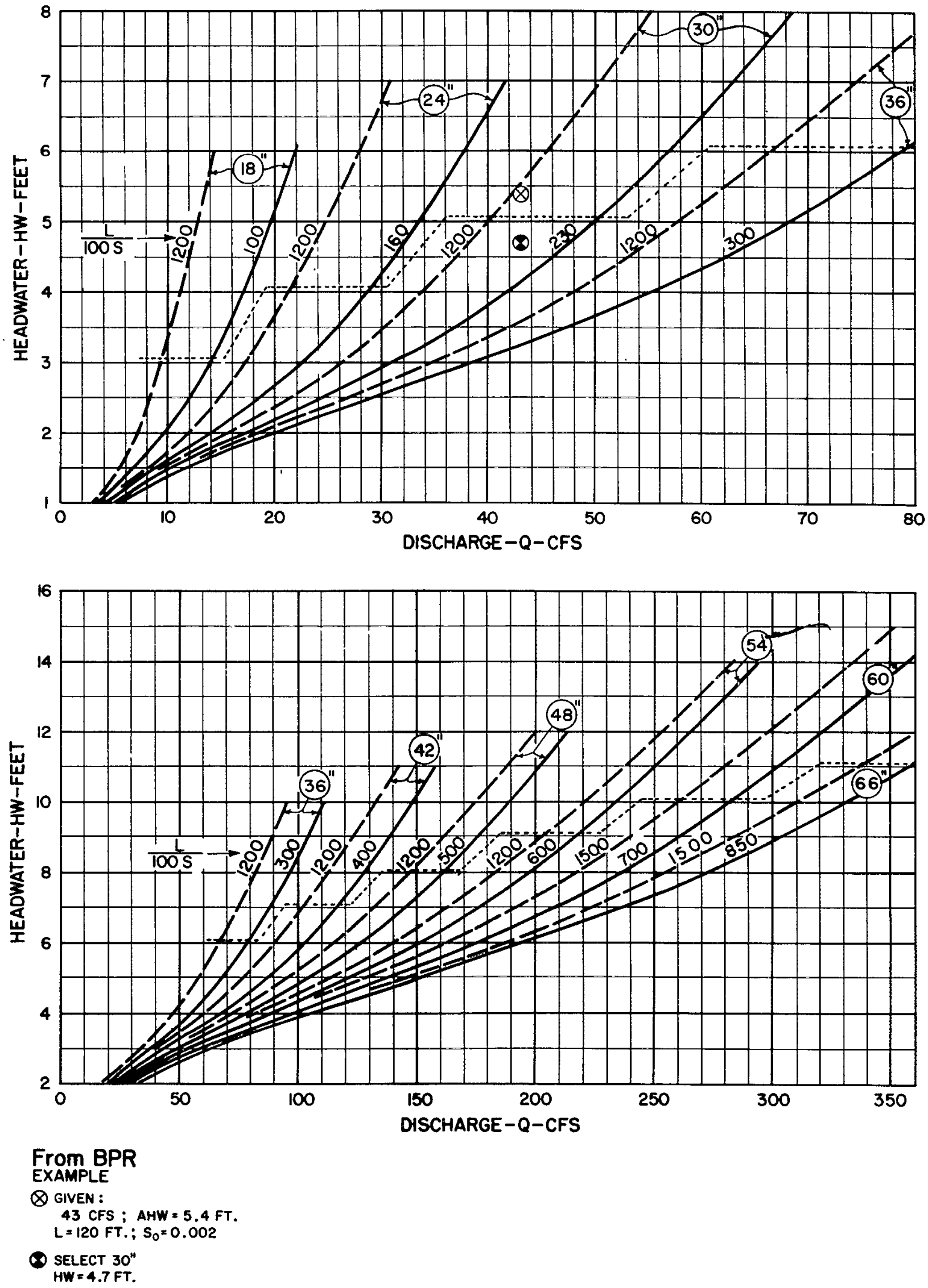


Figure 36—Culvert Capacity Circular Concrete Pipe Groove-Edged Entrance 18" to 66"

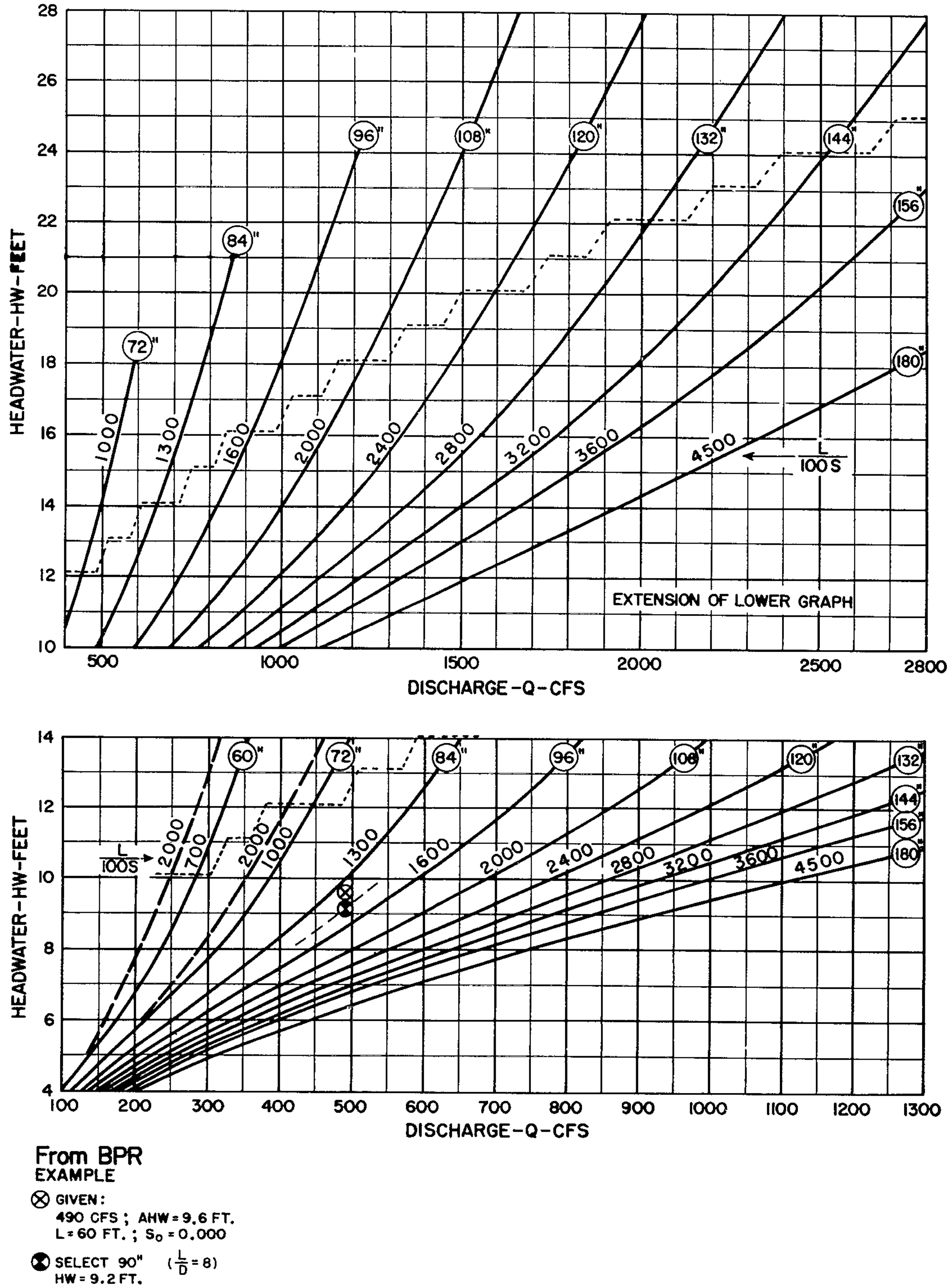


Figure 37—Culvert Capacity Circular Concrete Pipe Groove-Edged Entrance 60” to 180”

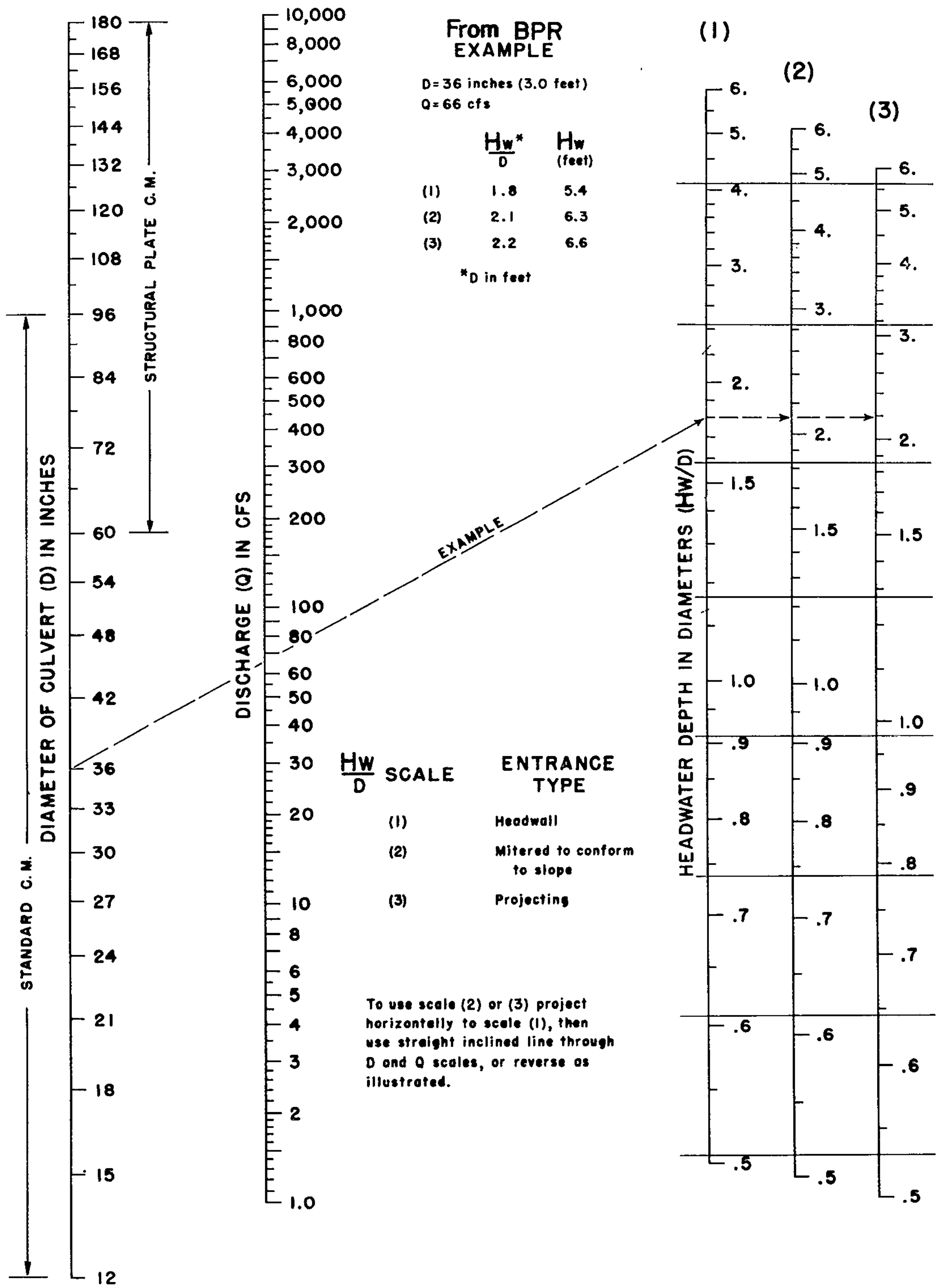


Figure 38—Headwater Depth for Corrugated Metal Pipe Culverts With Inlet Control

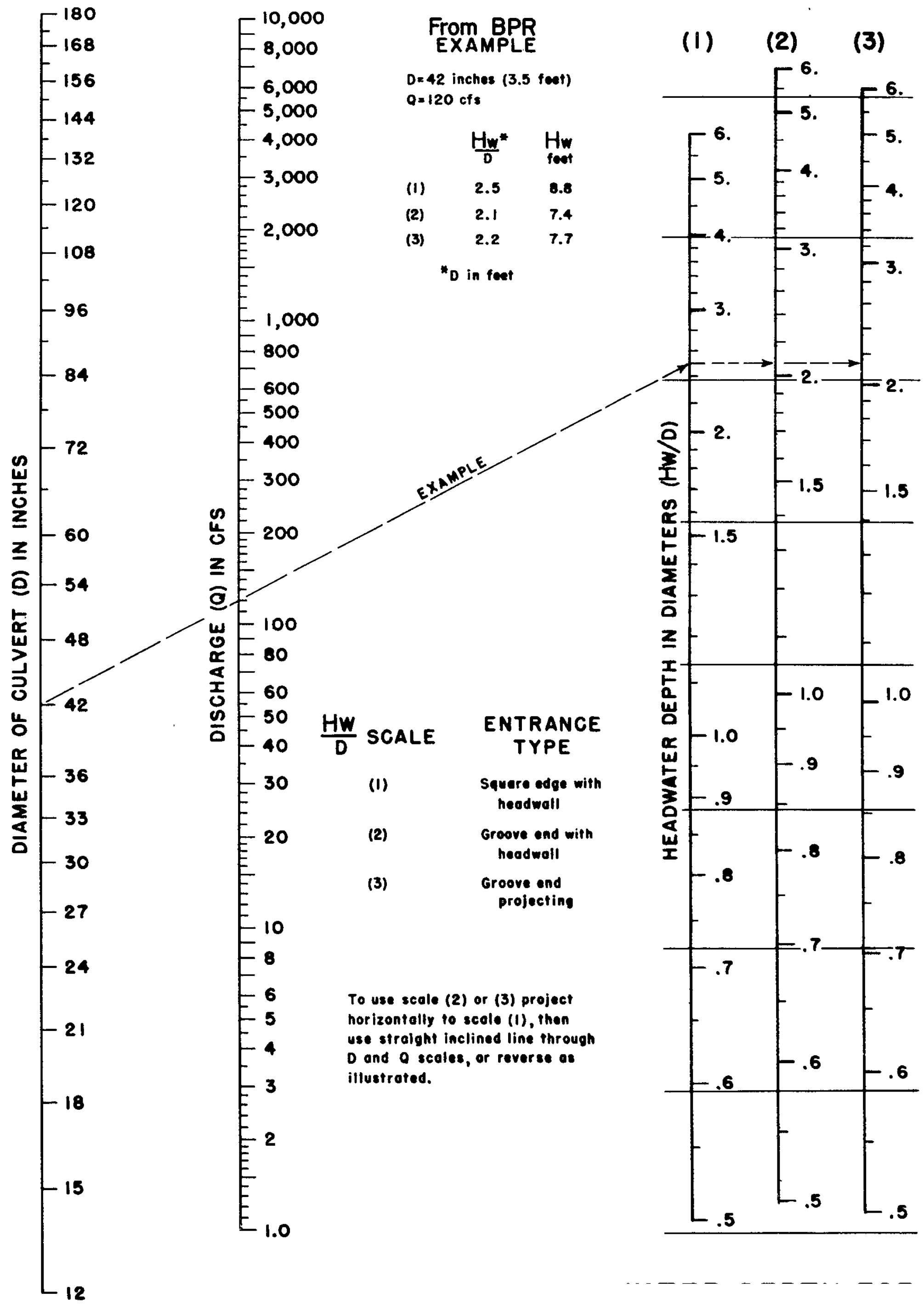
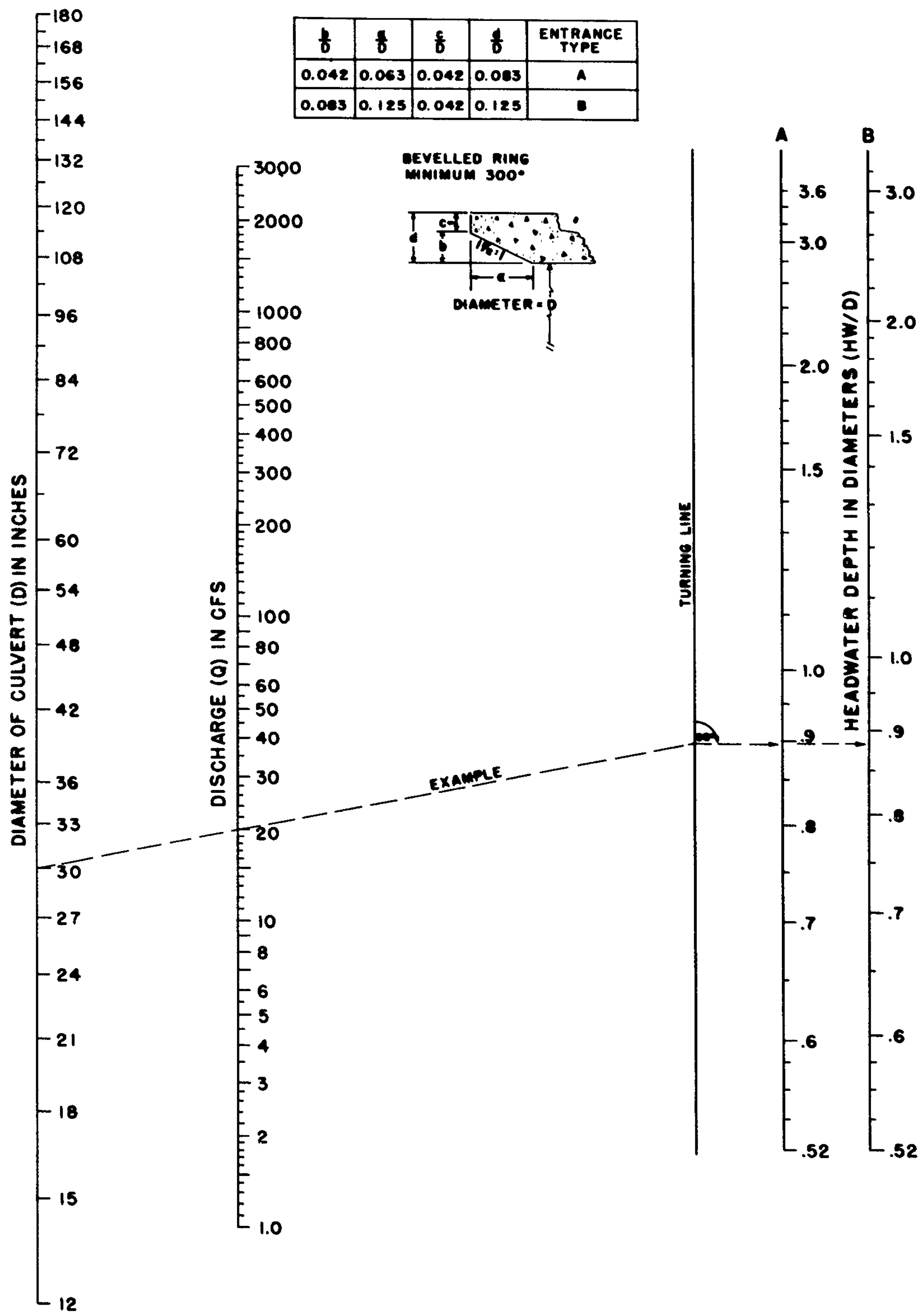


Figure 39—Headwater Depth for Concrete Pipe Culverts With Inlet Control



$\frac{b}{D}$	$\frac{a}{D}$	$\frac{c}{D}$	$\frac{d}{D}$	ENTRANCE TYPE
0.042	0.063	0.042	0.083	A
0.083	0.125	0.042	0.125	B

Figure 40—Headwater Depth for Circular Pipe Culverts With Beveled Ring Inlet Control

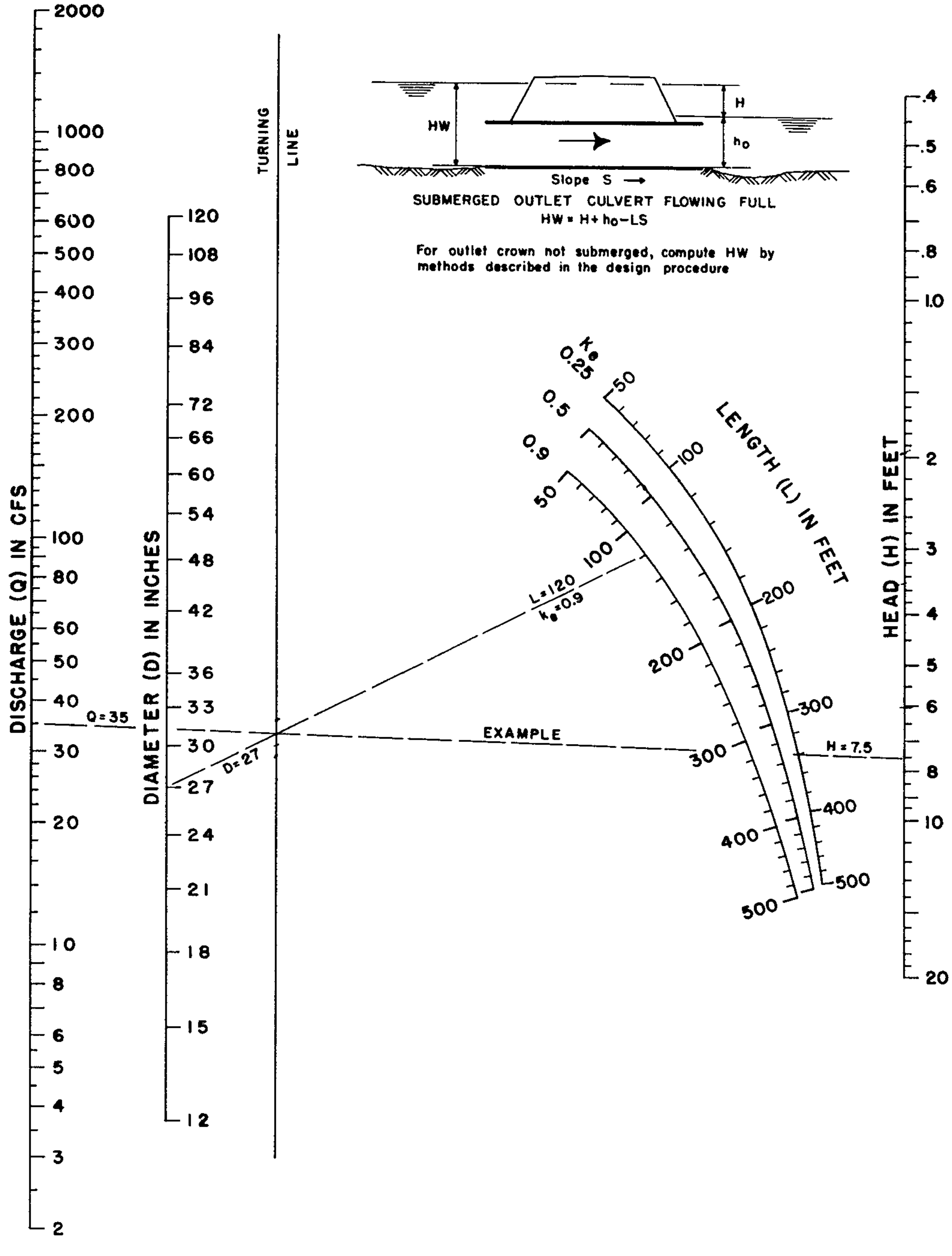


Figure 41—Head for Standard Corrugated Metal Pipe Culverts Flowing Full *n* = 0.024

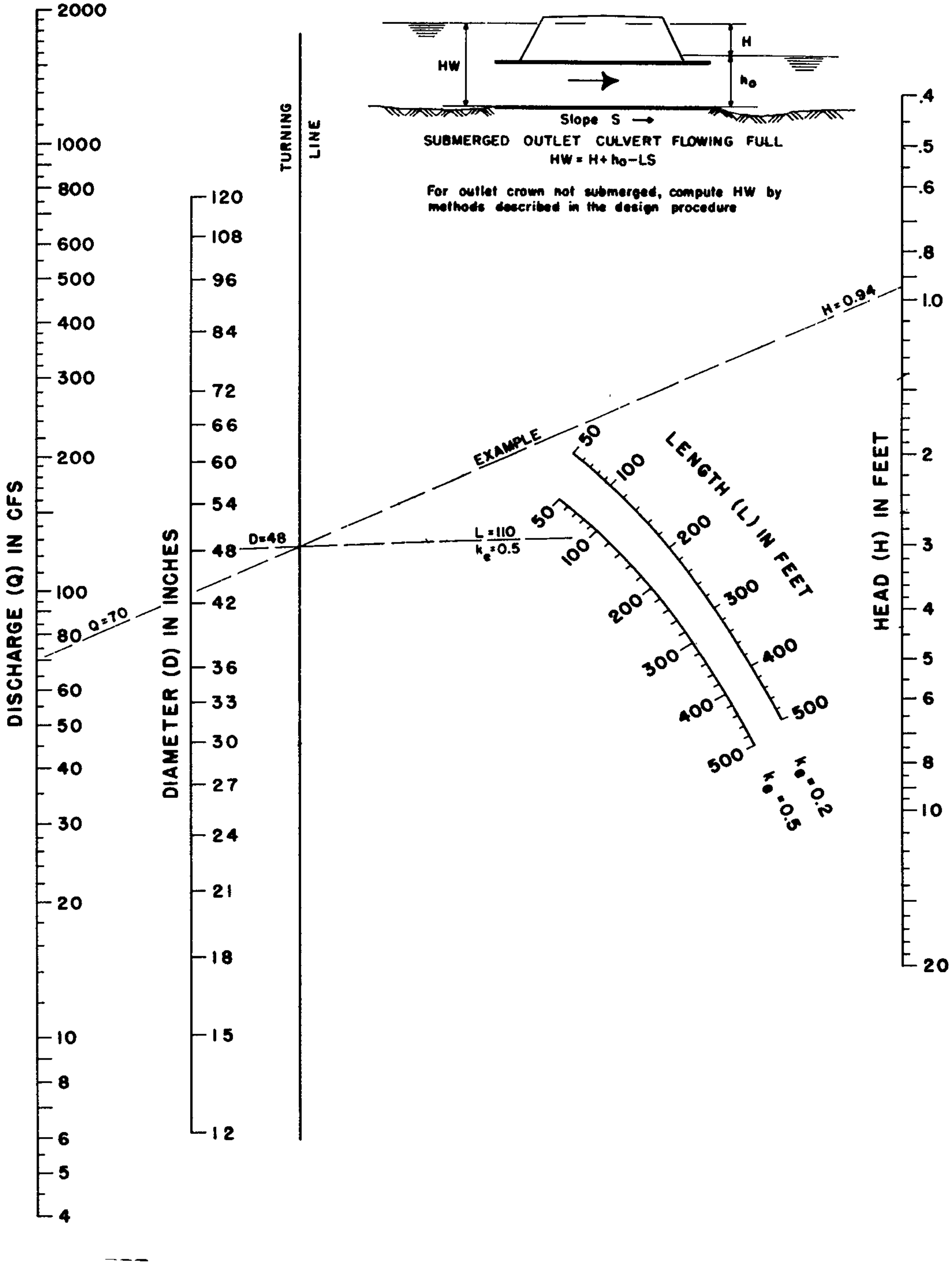


Figure 42—Head for Concrete Pipe Culverts Flowing Full *n* = 0.012

9.0 LARGE PIPES

Large pipes are often used as underground outfall conduits. An advantage of using pipes (circular conduits) rather than rectangular conduits is that pipes can withstand internal pressure to a greater degree than rectangular conduits can. Thus, the hydraulic design is not as critical, and a greater safety factor exists from the structural standpoint. Unless the designer is competent, experienced in open-channel hydraulics, and prepared to utilize laboratory model tests as a design aid, large pipes should be used rather than rectangular conduits. Cost differentials for the project should be carefully weighed before choosing the type of outfall conduit.

Disadvantages may include the fact that large pipes are less adaptable to an existing urban street where conflicts may exist with sanitary sewer pipes and other utilities.

9.1 Hydraulic Design

Large pipes are also considered as covered free-flow conduits; they are open channels with a cover (Steven, Simons, and Lewis 1971). Computational procedures for flow in large pipes are essentially the same as for canals and lined channels, except that consideration is given to diminishing capacity as the pipe flow nears the full depth.

Large pipes lend themselves to bends and slope changes more readily than do rectangular conduits. In a situation with a large pipe with the slope increasing in a downstream direction, there is no reason that the downstream pipe cannot be made smaller than the upstream pipe. However, the required transitional structure may rule out the smaller pipe from an economic standpoint. Improper necking down of large pipes has been a contributing factor in significant flooding of urban areas.

To aid in the solution of uniform flow computations for large pipes, see Table 3. Figures 43 and 44 are also useful aids for flow computations in pipes. Figure 45 is given as an additional design aid example.

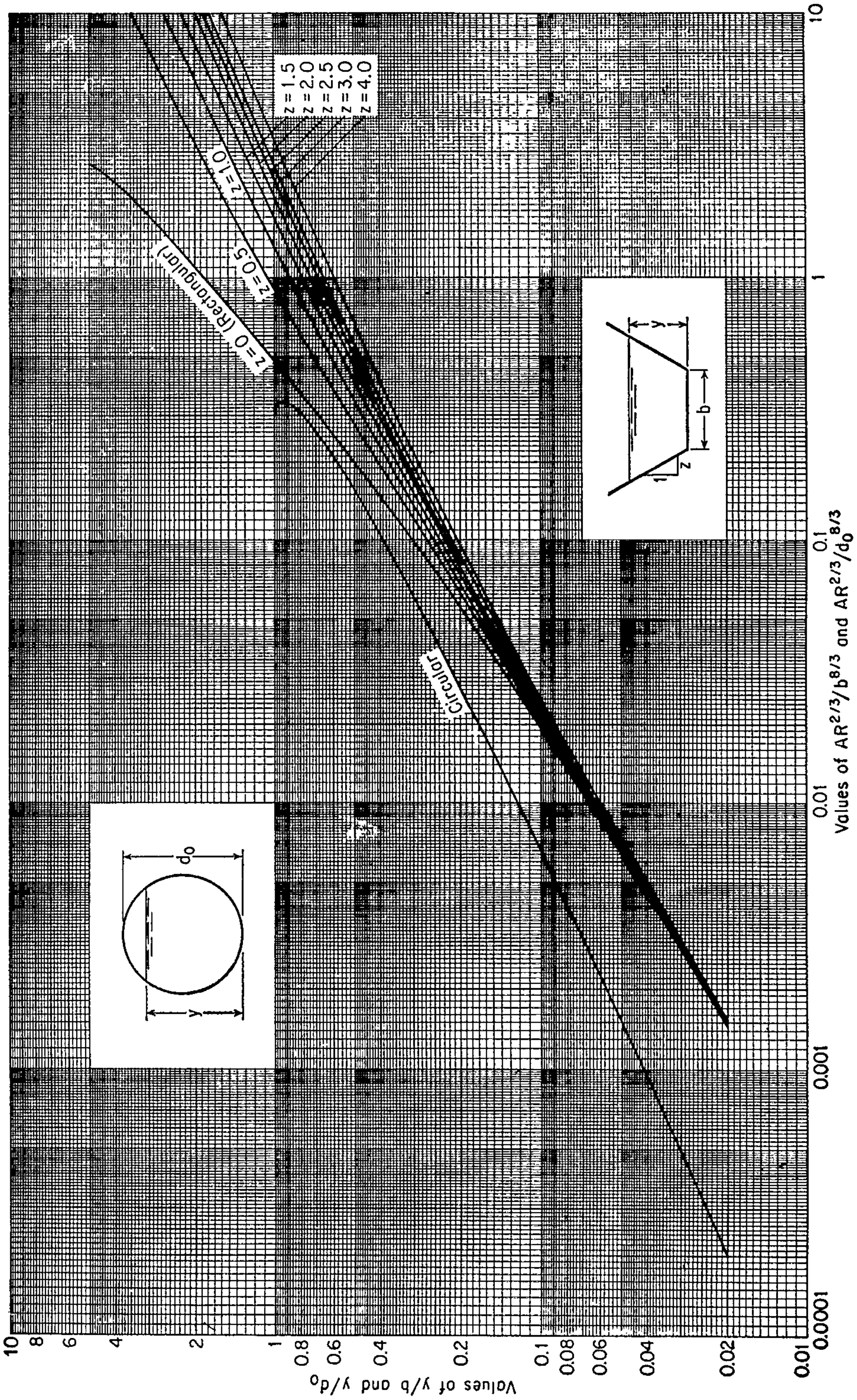


Figure 43—Normal Depth for Uniform Flow in Open Channels (Fletcher and Grace 1972)

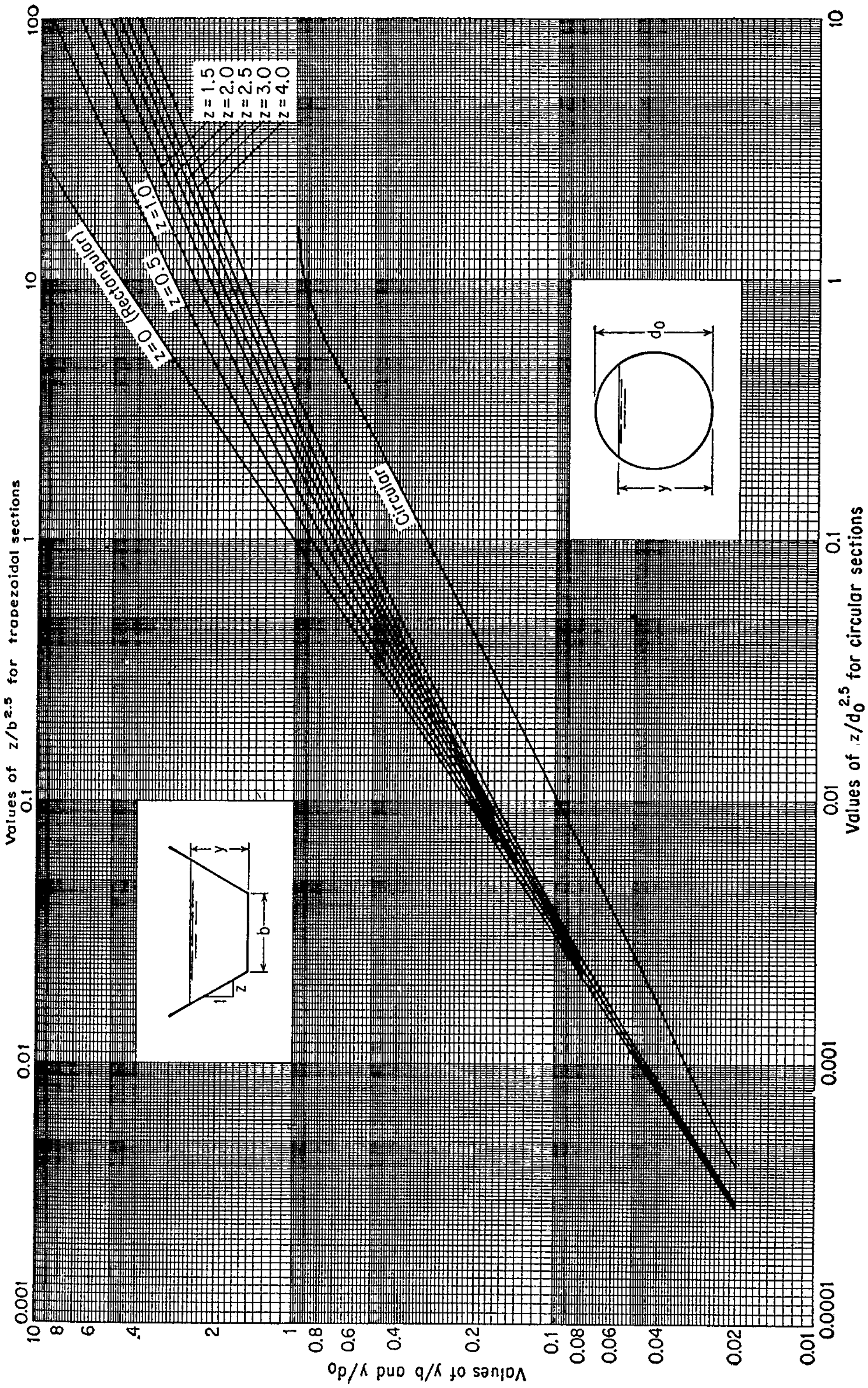


Figure 44—Curves for Determining the Critical Depth in Open Channels (Fletcher and Grace 1972)

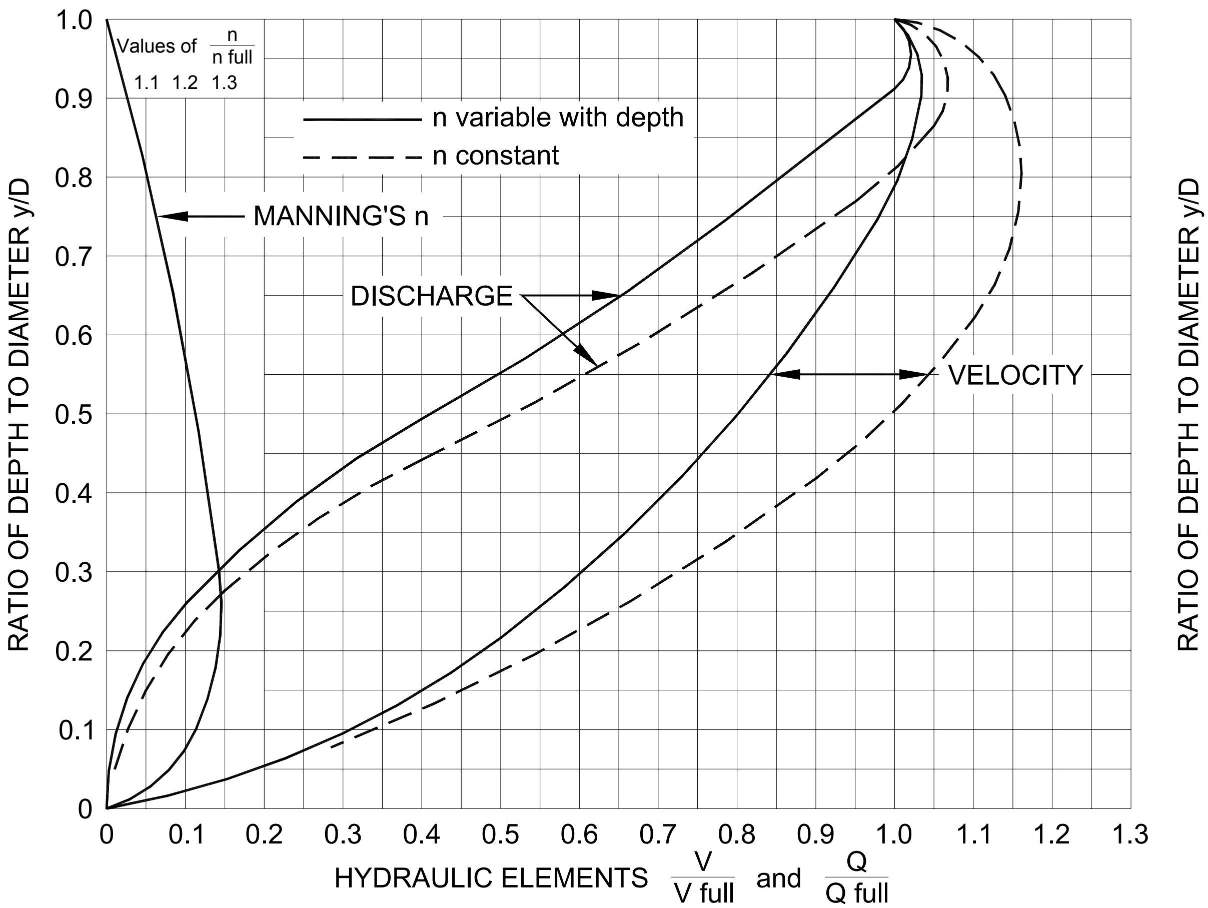


Figure 45—Hydraulic Properties of Pipes (Steven, Simons, and Lewis 1976)

Table 3
Uniform Flow in Circular Sections Flowing Partially Full
(Hipschman 1970)

y_0 = depth of flow
D = diameter of pipe
A = area of flow
R = hydraulic radius

Q = discharge in cfs by Manning formula
n = Manning coefficient
S_0 = slope of channel bottom and of the water surface

y_0/D	A/D^2	R/D	$Qn/(D^{8/3}S_0^{1/2})$	$Qn/(y_0^{8/3}S_0^{1/2})$	y_0/D	A/D^2	R/D	$Qn/(D^{8/3}S_0^{1/2})$	$Qn/(y_0^{8/3}S_0^{1/2})$
0.01	0.0013	0.0066	0.00007	15.040	0.51	0.4027	0.2531	0.23900	1.442
0.02	0.0037	0.0132	0.00031	10.570	0.52	0.4127	0.2562	0.24700	1.415
0.03	0.0069	0.0197	0.00074	8.560	0.53	0.4227	0.2592	0.25500	1.388
0.04	0.0105	0.0262	0.00138	7.380	0.54	0.4327	0.2621	0.26300	1.362
0.05	0.0147	0.0325	0.00222	6.550	0.55	0.4426	0.2649	0.27100	1.336
0.06	0.0192	0.0389	0.00328	5.950	0.56	0.4526	0.2676	0.27900	1.311
0.07	0.0242	0.0451	0.00455	5.470	0.57	0.4625	0.2703	0.28700	1.286
0.08	0.0294	0.0513	0.00604	5.090	0.58	0.4724	0.2728	0.29500	1.262
0.09	0.0350	0.0575	0.00775	4.760	0.59	0.4822	0.2753	0.30300	1.238
0.10	0.0409	0.0635	0.00967	4.490	0.60	0.4920	0.2776	0.31100	1.215
0.11	0.0470	0.0695	0.01181	4.250	0.61	0.5018	0.2799	0.31900	1.192
0.12	0.0534	0.0755	0.01417	4.040	0.62	0.5115	0.2821	0.32700	1.170
0.13	0.0600	0.0813	0.01674	3.860	0.63	0.5212	0.2842	0.33500	1.148
0.14	0.0668	0.0871	0.01952	3.690	0.64	0.5308	0.2862	0.34300	1.126
0.15	0.0739	0.0929	0.02250	3.540	0.65	0.5404	0.2882	0.35000	1.105
0.16	0.0811	0.0985	0.02570	3.410	0.66	0.5499	0.2900	0.35800	1.084
0.17	0.0885	0.1042	0.02910	3.280	0.67	0.5594	0.2917	0.36600	1.064
0.18	0.0961	0.1097	0.03270	3.170	0.68	0.5687	0.2933	0.37300	1.044
0.19	0.1039	0.1152	0.03650	3.060	0.69	0.5780	0.2948	0.38000	1.024
0.20	0.1118	0.1206	0.04060	2.960	0.70	0.5872	0.2962	0.38800	1.004
0.21	0.1199	0.1259	0.04480	2.870	0.71	0.5964	0.2975	0.39500	0.985
0.22	0.1281	0.1312	0.04920	2.790	0.72	0.6054	0.2987	0.40200	0.965
0.23	0.1365	0.1364	0.05370	2.710	0.73	0.6143	0.2998	0.40900	0.947
0.24	0.1449	0.1416	0.05850	2.630	0.74	0.6231	0.3008	0.41600	0.928
0.25	0.1535	0.1466	0.06340	2.560	0.75	0.6319	0.3017	0.42200	0.910
0.26	0.1623	0.1516	0.06860	2.490	0.76	0.6405	0.3024	0.42900	0.891
0.27	0.1711	0.1566	0.07390	2.420	0.77	0.6489	0.3031	0.43500	0.873
0.28	0.1800	0.1614	0.07930	2.360	0.78	0.6573	0.3036	0.44100	0.856
0.29	0.1890	0.1662	0.08490	2.300	0.79	0.6655	0.3039	0.44700	0.838
0.30	0.1982	0.1709	0.09070	2.250	0.80	0.6736	0.3042	0.45300	0.821
0.31	0.2074	0.1756	0.09660	2.200	0.81	0.6815	0.3043	0.45800	0.804
0.32	0.2167	0.1802	0.10270	2.140	0.82	0.6893	0.3043	0.46300	0.787
0.33	0.2260	0.1847	0.10890	2.090	0.83	0.6969	0.3041	0.46800	0.770
0.34	0.2355	0.1891	0.11530	2.050	0.84	0.7043	0.3038	0.47300	0.753
0.35	0.2450	0.1935	0.12180	2.000	0.85	0.7115	0.3033	0.47700	0.736
0.36	0.2546	0.1978	0.12840	1.958	0.86	0.7186	0.3026	0.48100	0.720
0.37	0.2642	0.2020	0.13510	1.915	0.87	0.7254	0.3018	0.48500	0.703
0.38	0.2739	0.2062	0.14200	1.875	0.88	0.7320	0.3007	0.48800	0.687
0.39	0.2836	0.2102	0.14900	1.835	0.89	0.7384	0.2995	0.49100	0.670
0.40	0.2934	0.2142	0.15610	1.797	0.90	0.7445	0.2980	0.49400	0.654
0.41	0.3032	0.2182	0.16330	1.760	0.91	0.7504	0.2963	0.49600	0.637
0.42	0.3130	0.2220	0.17050	1.724	0.92	0.7560	0.2944	0.49700	0.621
0.43	0.3229	0.2258	0.17790	1.689	0.93	0.7612	0.2921	0.49800	0.604
0.44	0.3328	0.2295	0.18540	1.655	0.94	0.7662	0.2895	0.49800	0.588
0.45	0.3428	0.2331	0.19290	1.622	0.95	0.7707	0.2865	0.49800	0.571
0.46	0.3527	0.2366	0.20100	1.590	0.96	0.7749	0.2829	0.49600	0.553
0.47	0.3627	0.2401	0.20800	1.559	0.97	0.7785	0.2787	0.49400	0.535
0.48	0.3727	0.2435	0.21600	1.530	0.98	0.7817	0.2735	0.49800	0.517
0.49	0.3827	0.2468	0.22400	1.500	0.99	0.7841	0.2666	0.48300	0.496
0.50	0.3927	0.2500	0.23200	1.471	1.00	0.7854	0.2500	0.46300	0.463

9.1.1 Entrance

The longer a pipe is, the more important is design of the entrance. A large pipe unable to flow at the design capacity represents wasted investment. Acceleration of flow, typically to the design velocity of the pipe reach immediately downstream, is often an important characteristic of the entrance. Air vents are necessary downstream of the entrance to allow entrained air to escape and to act as breathers.

9.1.2 Internal Pressure

The allowable internal pressure is limited by the structural design of the pipe; however, it is not as critical as with rectangular conduits, with up to perhaps 25 feet of head being permissible in some pipe designs before failure commences. It is evident, however, that large pipe outfalls cannot be designed for flow under any significant pressure because then inflow from other lines could not enter, and water would flow out of storm inlets rather than into these inlets. The internal pressure aspect is important only as a safety factor in the event of a choking of capacity or an inadvertent flow surcharge.

9.1.3 Curves and Bends

Curves and bends are permitted, but detailed analysis is required to ensure structural integrity and proper hydraulic functioning of the conduit. Maintenance access should be provided in the proximity of all bends. Hydraulic analyses are important at locations where hydraulic jumps may occur.

9.1.4 Transitions

Transitions are discussed Section 10.1.4.

9.1.5 Air Entrainment

The reader is referred to Section 10.1.5.

9.1.6 Major Inlets

Inflow to the conduit can cause unanticipated hydraulic variations; however, the analytical approach need not be as rigorous as with rectangular conduits.

9.2 Appurtenances

The reader is referred to Section 10.2.

9.3 Safety

See guidance in Section 4.6.

10.0 RECTANGULAR CONDUITS

The use of rectangular conduits of larger capacity can sometimes have cost advantages over large-diameter pipe. Furthermore, because they can be poured in place, advantages accrue in being able to incorporate conflicting utilities into the floor and roof of the structure.

Major disadvantages of rectangular conduits as storm sewers are:

1. The conduit's capacity drops significantly when the water surface reaches its roof since the wetted perimeter dramatically increases. The drop is 20% for a square cross section and more for a rectangular cross section where the width is greater than the height.

2. Normal structural design, because of economics, usually does not permit any significant interior pressures, meaning that if the conduit reached a full condition and the capacity dropped, there could be a failure due to interior pressures caused by a choking of the capacity (Murphy 1971).

It is apparent that the use of long rectangular conduits for outfall purposes requires a high standard of planning and design involving complex hydraulic considerations.

10.1 Hydraulic Design

Rectangular conduits are often considered as a covered free-flow conduit. They are open channels with a cover (Smith 1974). Computational procedures for flow in rectangular conduits are essentially the same as for canals and lined channels, except that special consideration is needed in regard to rapidly increasing flow resistance when a long conduit becomes full.

An obstruction, or even a confluence with another conduit, may cause the flow in a near-full rectangular conduit to strike the roof and choke the capacity. The capacity reduction may then cause the entire upstream reach of the conduit to flow full, with a resulting surge and pressure head increase of sufficient magnitude to cause a structural failure. Thorough design is required to overcome this inherent potential problem. Structural design must account for internal pressure if pressure will exist.

Structural requirements and efficiency for sustaining external loads, rather than hydraulic efficiency, usually control the shape of the rectangular conduit. In urban drainage use, a rectangular conduit should usually have a straight alignment and should not decrease in size or slope in a downstream direction. It is desirable to have a slope that increases in a downstream direction as an added safety factor against it flowing full. This is particularly important for supercritical velocities that often exist in long conduits. For flatter-sloped conduits, the sediment deposition problem must be considered to prevent an inadvertent loss of capacity.

Roughness coefficients (Table 4) should be chosen carefully because of their effect on proper operation of the conduit. Quality control is important during construction; attention must be paid to grinding off projections and keeping good wall alignment. When using precast box sections, joint alignment and grouting are especially important.

Bedding and cover on conduits are structural considerations, and specifications for bedding and cover are closely allied to the loads and forces used in the structural design.

Table 4
Roughness Coefficients for Large Concrete Conduits

Type of Concrete Conduit	Roughness Coefficient
Precast concrete pipe, good joint alignment	0.012
Precast concrete pipe, ordinary joint alignment	0.013
Poured-in-place steel forms, projections 1/8” or less	0.013
Poured-in-place smooth wood forms, projections 1/8” or less	0.013
Poured-in-place ordinary work with steel forms	0.014
Poured-in-place ordinary work with wood forms	0.015

10.1.1 Entrance

Because a long rectangular conduit is costly, as well as for other reasons, the hydraulic characteristics at the entrance are particularly important. A conduit that cannot flow at the design discharge because of an inadequate or clogged inlet represents wasted investment and can result in flooding of homes, buildings, structures, and other urban infrastructure.

The entrances take on a special degree of importance for rectangular conduits, however, because the flow must be limited to an extent to ensure against overcharging the conduit. Special maximum-flow limiting entrances are often used with rectangular conduits. These special entrances should reject flow over the design discharge so that, if a runoff larger than the design flow occurs, the excess water will flow via other routes, often overland. A combined weir-orifice design is useful for this purpose. Model tests are needed for dependable design (Murphy 1971).

A second function of the entrance should be to accelerate the flow to the design velocity of the conduit, usually to meet the velocity requirements for normal depth of flow in the upstream reach of the conduit.

Air vents are needed at regular intervals to obviate both positive and negative pressures and to permit released entrained air to readily escape from the conduit.

10.1.2 Internal Pressure

The allowable internal pressure in a rectangular conduit is limited by structural design. Often, internal pressures are limited to no more than 2 to 4 feet of head before structural failure will commence, if structural design has not been based on internal pressure. Surges or conduit capacity choking cannot normally be tolerated.

10.1.3 Curves and Bends

The analysis of curves in rectangular conduits is critical from the point of view of the water reaching the roof of the conduit and related hydraulic losses. Superelevation of the water surface must also be studied, and allowances must be made for a changing hydraulic radius, particularly in high-velocity flow. Dynamic loads created by the curves must be analyzed to assure structural integrity for the maximum flows.

10.1.4 Transitions

Transitions provide complex hydraulic problems and require specialized analyses. Transitions, either contracting or expanding, are important with most large outfall conduits because of high-velocity flow. The development of shock waves that continue downstream can create significant problems in regard to proper conduit functioning. The best way to study transitions is through model tests (Fletcher and Grace 1972). Analytical procedures can only give approximate results. Poor transitions can cause upstream problems with both subcritical and supercritical flow, as well as unnecessary flooding.

10.1.5 Air Entrainment

Entrained air causes a swell in the volume of water and an increase in depth than can cause flow in the conduit to reach the height of the roof with resulting loss of capacity; therefore, hydraulic design must account for entrained air. In rectangular conduits and circular pipes, flowing water will entrain air at velocities of about 20 ft/sec and higher. Additionally, other factors such as entrance condition, channel roughness, distance traveled, channel cross section, and volume of discharge all have some bearing on air entrainment. Volume swell can be as high as 20% (Hipschman 1970).

10.1.6 Major Inlets

Major inlets to a rectangular conduit at junctions or large storm inlets should receive a rigorous hydraulic analysis to assure against mainstream conduit flow striking the top of the rectangular conduit due to momentum changes in the main flow body as a result of the introduction of additional flow. Model tests may be necessary.

10.1.7 Sedimentation

The conduit must be designed to obviate sediment deposition problems during storm runoff events that have a frequency of occurrence of about twice each year. That is, at least twice per year, on average, the storm runoff velocity should be adequate to scour deposited sediment from the box section.

10.2 Appurtenances

The appurtenances to a long rectangular conduit are dictated by the individual needs of the particular project. Most appurtenances have some effect upon the overall operation of the system; the designer must consider all of these effects.

10.2.1 Energy Dissipators

Long conduits usually have high exit velocities that must be slowed to avoid downstream problems and damage. Energy dissipators are nearly always required.

10.2.2 Access Manholes

A long rectangular conduit should be easy to inspect, and, therefore, access manholes are desirable at various locations. If a rectangular conduit is situated under a curb, the access manholes may be combined with the storm sewer system inlets. Manholes should be aligned with the vertical wall of the box to allow rungs in the riser and box to be aligned.

Access manholes and storm inlets are useful for permitting air to flow in and out of a rectangular conduit as filling and emptying of the conduit occurs. They might also be considered safety water ejection ports should the conduit ever inadvertently flow full and cause a pileup of water upstream. The availability of such ejection ports could very well save a rectangular conduit from serious structural damage.

10.2.3 Vehicle Access Points

A large rectangular conduit with a special entrance and an energy dissipater at the exit may need an access hole for vehicle use in case major repair work becomes necessary. A vehicle access point might be a large, grated opening just downstream from the entrance. This grated opening can also serve as an effective air breather for the conduit. Vehicles may be lowered into the conduit by a crane or A-frame.

10.2.4 Safety

See discussion on public safety design consideration in Section 4.6.

11.0 BRIDGES

There are extensive manuals on bridges that are available and should be used in bridge hydraulic studies and river stability analysis. Some of the best include:

1. *Hydraulics of Bridge Waterways* Hydraulic Design Series No. 1(FHWA 1978). This is a good basic reference.

2. *Highway in the River Environment* (Richardson 1988 draft with appendices and 1974). This is particularly good for hydraulics, geomorphology, scour, and degradation.

3. *Design Manual for Engineering Analysis of Fluvial Systems for the Arizona Department of Water Resources* (LSA 1985). This is a prime reference on hydraulics and the three-level sediment transport analysis, with examples.

Photograph 7—A stable channel at bridges is important and includes caring for the stream downstream of the bridge.

4. *Hydraulic Analysis Location and Design of Bridges* Volume 7 (AASHTO 1987). This is a good overview document.

5. *Technical Advisory on Scour at Bridges* (FHWA 1988). This presents information similar to references 2, 3, and 4 above, but in a workbook format, and perhaps oversimplified.

Bridges are required across nearly all open urban channels sooner or later and, therefore, sizing the bridge openings is of paramount importance. Open channels with improperly designed bridges will either have excessive scour or deposition or not be able to carry the design flow.

11.1 Basic Criteria

Bridge openings should be designed to have as little effect on the flow characteristics as reasonable, consistent with good bridge design and economics. However, in regard to supercritical flow with a lined channel, the bridge should not affect the flow at all—that is, there should be no projections into the design water prism.

11.1.1 Design Approach

The method of planning for bridge openings must include water surface profiles and hydraulic gradient analyses of the channel for the major storm runoff. Once this hydraulic gradient is established without the bridge, the maximum reasonable effect on the channel flow by the bridge should be determined. In urban cases this should not exceed a backwater effect of more than 6 to 12 inches.

Velocities through the bridge and downstream of the bridge must receive consideration in choosing the bridge opening. Velocities exceeding those permissible will require special protection of the bottom and banks.

For supercritical flow, the clear bridge opening should permit the flow to pass under unimpeded and unchanged in cross section.

11.1.2 Bridge Opening Freeboard

The distance between the design flow water surface and the bottom of the bridge deck will vary from case to case. However, the debris that may be expected must receive full consideration in setting the freeboard. Freeboard may vary from several feet to minus several feet. There are no general rules. Each case must be studied separately.

Bridges that are securely anchored to foundations and designed to withstand the dynamic forces of the flowing water might, in some cases, be designed without freeboard.

11.1.3 Hydraulic Analysis

The hydraulic analysis procedures described below are suitable, although alternative methods such as FHWA HY-4 or HEC-RAS are acceptable, as well.

The design of a bridge opening generally determines the overall length of the bridge. The length affects the final cost of the bridge. The hydraulic engineering in the design of bridges has more impact on the bridge cost than does the structural design. Good hydraulic engineering is necessary for good bridge design (FHWA 1978, Richardson 1974 and 1988).

The reader is referred to *Hydraulics of Bridge Waterways* (U.S. Bureau of Public Roads 1978) for more guidance on the preliminary assessment approach described below. In working with bridge openings, the designer may use the designation shown in Figure 46.

11.1.4 Expression for Backwater

A practical expression for backwater has been formulated by applying the principle of conservation of energy between the point of maximum backwater upstream from the bridge and a point downstream from the bridge at which normal stage has been reestablished, as shown in Sections 1 and 4, respectively, of Figure 46. The expression is reasonably valid if the channel in the vicinity of the bridge is reasonably uniform, the gradient of the bottom is approximately constant between Sections 1 and 4, there is no appreciable erosion of the bed in the constriction due to scour, and the flow is subcritical.

The expression for computation of backwater upstream from a bridge constricting the flow is as follows:

$$h_1^* = (K*)\left(\frac{(V_{n2})^2}{2g}\right) + \propto 1\left[\left(\frac{A_{n2}}{A_4}\right)^2 - \left(\frac{A_{n2}}{A_1}\right)^2\right]\frac{V_{n2}{}^2}{2g}$$

(Equation 29)

in which:

h_1^* = total backwater (ft)

K^* = total backwater coefficient

$\propto 1 = \dfrac{qv^2}{QV_1^2}$ = kinetic energy coefficient

A_{n2} = gross water area in constriction measured below normal stage (ft^2)

V_{n2} = average velocity in constriction or Q/A_{n2} (ft/sec). The velocity V_{n2} is not an actual measurable velocity but represents a reference velocity readily computed for both model and field structures.

A_4 = water area at Section 4 where normal stage is reestablished (ft^2)

A_1 = total water area at Section 1 including that produced by the backwater (ft^2)

g = acceleration of gravity (32.2 ft/sec^2)

To compute backwater by Equation 29, it is necessary to obtain the approximate value of h_1^* by using the first part of the equation:

$$h_1^* = (K*)\left(\frac{V_{n2}^2}{2g}\right)$$

(Equation 30)

The value of A_1 in the second part of Equation 29, which depends on h_1^*, can then be determined.

This part of the expression represents the difference in kinetic energy between Sections 4 and 1, expressed in terms of the velocity head $\frac{V_{n2}^2}{2g}$. Equation 30 may appear cumbersome, but it was set up as shown to permit omission of the second part when the difference in kinetic energy between Sections 4 and 1 is small enough to be insignificant in the final result.

To permit the designer to readily recognize cases in which the kinetic energy term may be ignored, the following guides are provided:

$M > 0.7$, where M = bridge opening ratio

$V_{n2} < 7$ ft/sec

$(K*)\left(\frac{V_{n2}^2}{2g}\right) < 0.5$ ft

If values meet all three conditions, the backwater obtained from Equation 30 can be considered sufficiently accurate. Should one or more of the values not meet the conditions set forth, it is advisable to use Equation 29 in its entirety. The use of the guides is further demonstrated in the examples given in FHWA (1978) that should be used in all bridge design work.

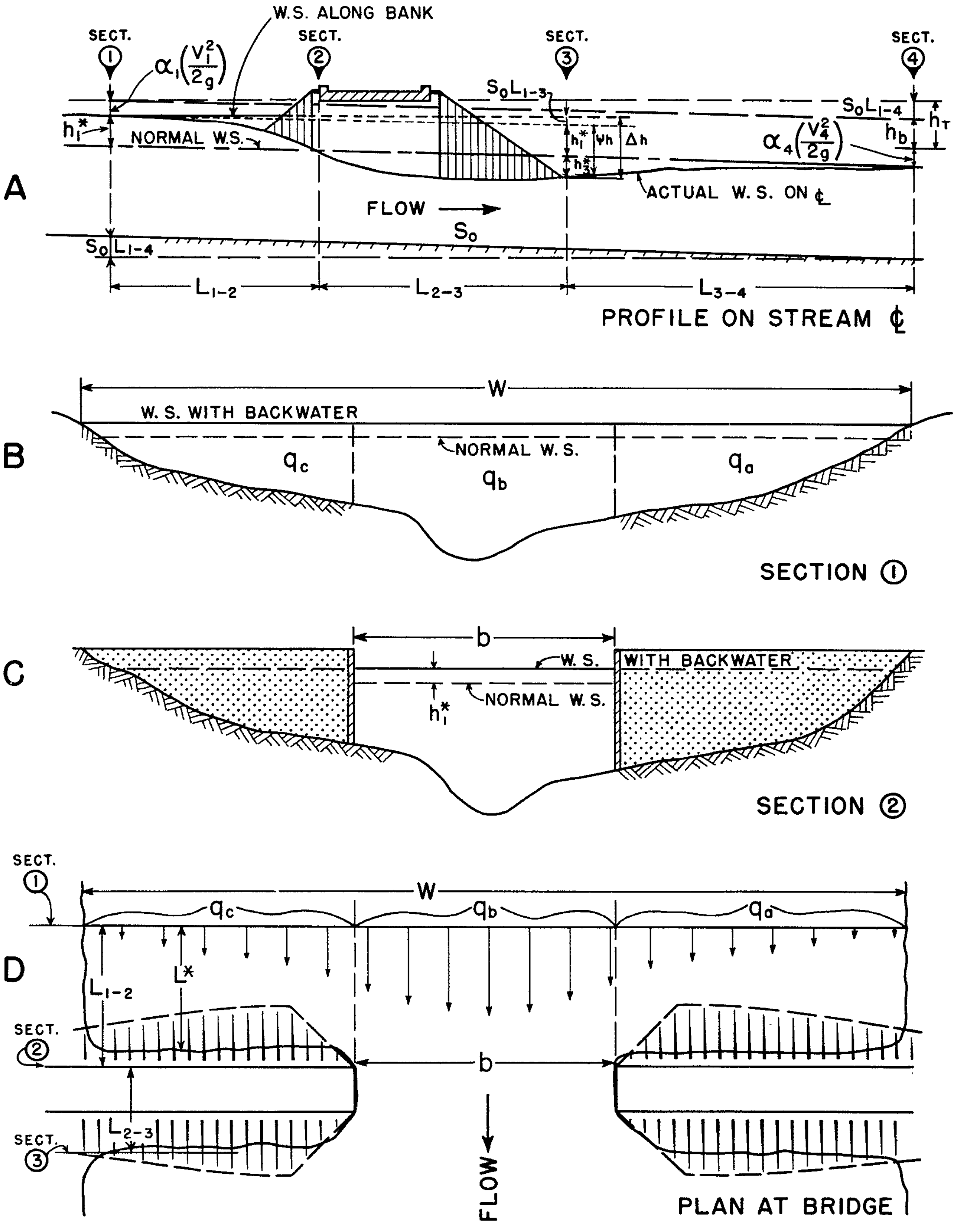


Figure 46—Normal Bridge Crossing Designation

11.1.5 Backwater Coefficient

The value of the overall backwater coefficient K^*, which was determined experimentally, varies with:

1. Stream constriction as measured by bridge opening ratio, M.
2. Type of bridge abutment: wingwall, spill through, etc.
3. Number, size, shape, and orientation of piers in the constriction.
4. Eccentricity, or asymmetric position of bridge with the floodplains.
5. Skew (bridge crosses floodplain at other than 90 degree angle).

The overall backwater coefficient K^* consists of a base curve coefficient, K_b, to which are added incremental coefficients to account for the effect of piers, eccentricity, and skew. The value of K^* is primarily dependent on the degree of constriction of the flow but also changes to a limited degree with the other factors.

11.1.6 Effect of *M* and Abutment Shape (Base Curves)

Figure 47 shows the base curve for backwater coefficient, K_b, plotted with respect to the opening ratio, M, for several wingwall abutments and a vertical wall type. Note how the coefficient K_b increases with channel constriction. The several curves represent different angles of wingwalls as can be identified by the accompanying sketches; the lower curves represent the better hydraulic shapes.

Figure 48 shows the relation between the backwater coefficient, K_b, and M for spill-through abutments for three embankment slopes. A comparison of the three curves indicates that the coefficient is little affected by embankment slope. Figures 47 and 48 are “base curves” and K_b is referred to as the “base curve coefficient.” The base curve coefficients apply to normal crossings for specific abutment shapes but do not include the effect of piers, eccentricity, or skew.

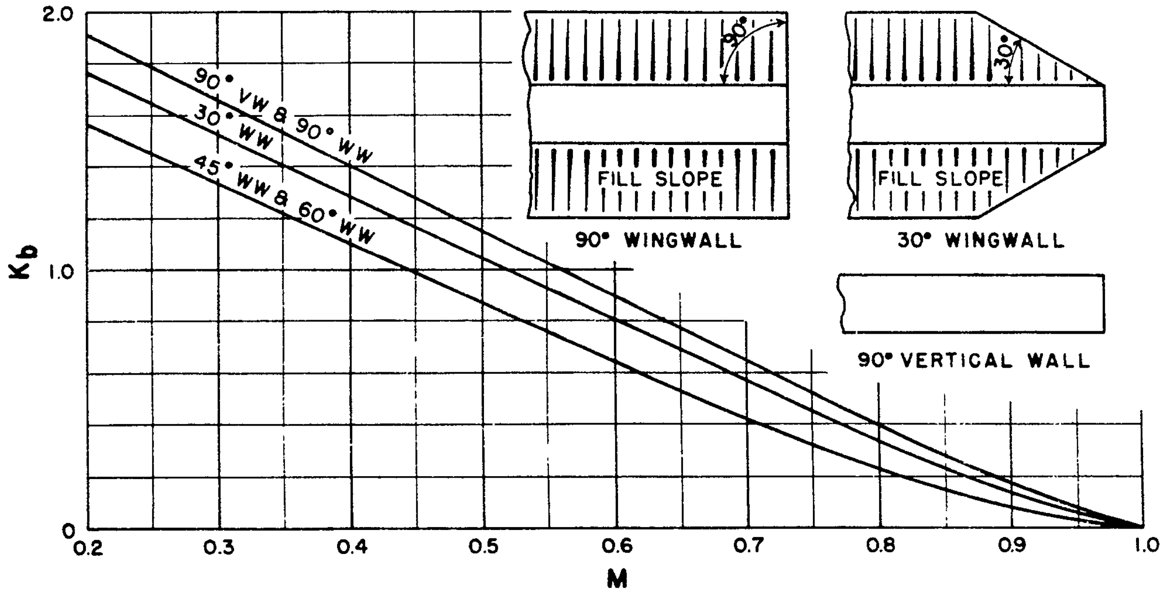


Figure 47—Base Curves for Wingwall Abutments

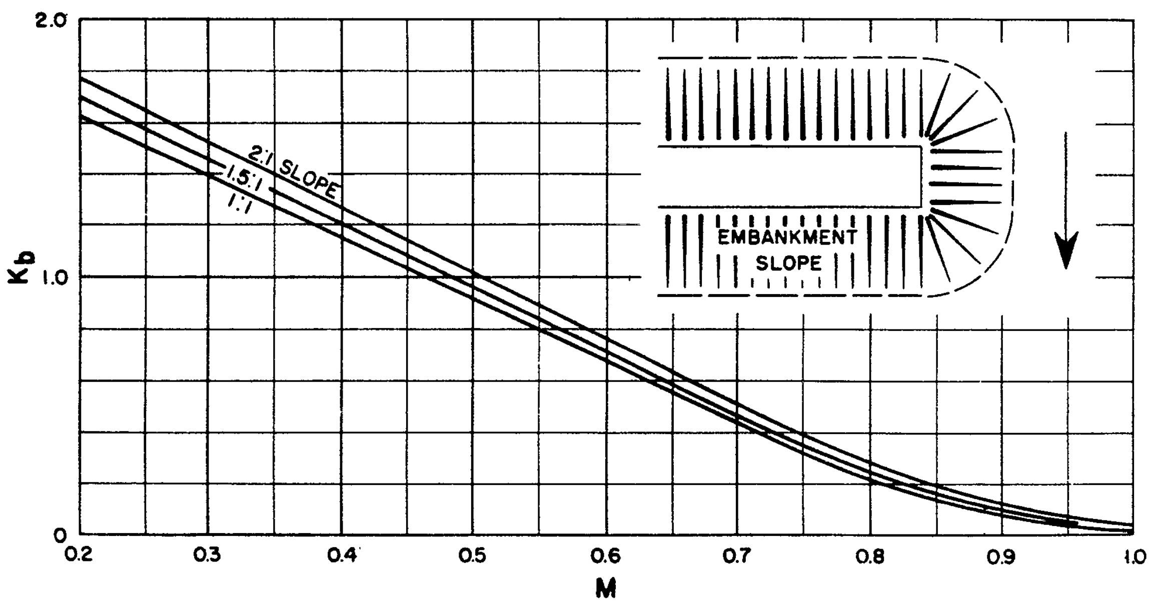


Figure 48—Base Curves for Spillthrough Abutments

11.1.7 Effect of Piers (Normal Crossings)

The effect on the backwater from introduction of piers in a bridge constriction has been treated as an incremental backwater coefficient designated ΔK_p, which is added to the base curve coefficient when piers are a factor. The value of the incremental backwater coefficient, ΔK_p, is dependent on the ratio that the area of the piers bears to the gross area of the bridge opening, the type of piers (or piling in the case of pile bents), the value of the bridge opening ratio, M, and the angularity of the piers with the direction of flood flow. The ratio of the water area occupied by piers, A_p, to the gross water area of the constriction, A_{n2}, both based on the normal water surface, has been assigned the letter J. In computing the gross water area, A_{n2}, the presence of piers in the constriction is ignored. The incremental backwater coefficient for the more common types of piers and pile bents can be obtained from Figure 49. The procedure is to enter Chart A, Figure 49, with the proper value of J and read ΔK and obtain the correction factor σ from Chart B, Figure 49, for opening ratios other than unity. The incremental backwater coefficient is then

$$\Delta K_p = \Delta K \sigma$$

(Equation 31)

The incremental backwater coefficients for piers can, for all practical purposes, be considered independent of diameter, width, or spacing but should be increased if there are more than 5 piles in a bent. A bent with 10 piles should be given a value of ΔK_p about 20% higher than those shown for bents with 5 piles. If there is a good possibility of trash collecting on the piers, it is advisable to use a value greater than the pier width to include the trash. For a normal crossing with piers, the total backwater coefficient becomes:

$$K^* = K_b \text{ (Figures 47or 48)} + \Delta K_p \text{ (Figure 49)}$$

(Equation 32)

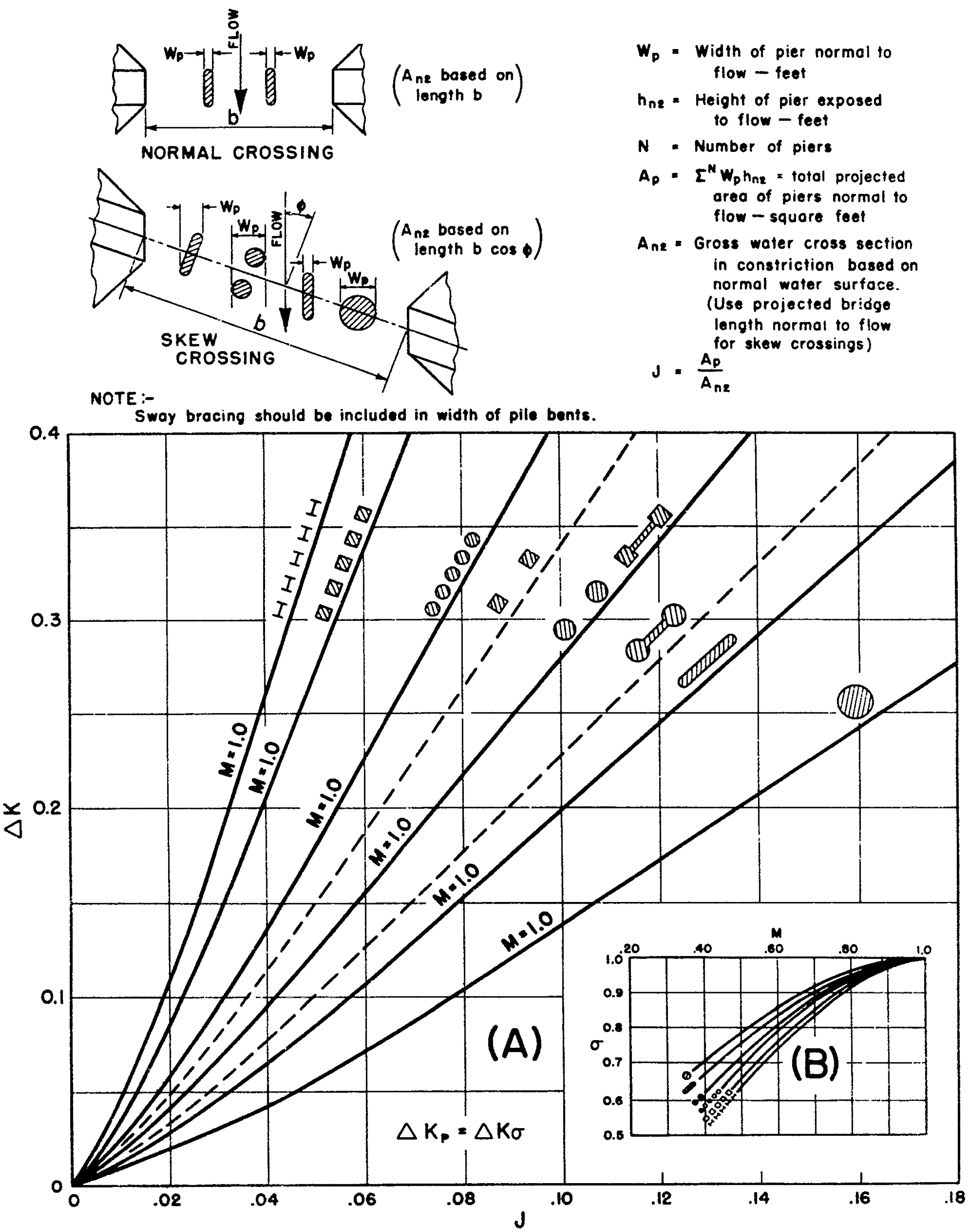


Figure 49—Incremental Backwater Coefficient for Pier

11.2 Design Procedure

The following is a brief step-by-step outline for determination of backwater produced by a bridge constriction:

1. Determine the magnitude and frequency of the discharge for which the bridge is to be designed.

2. Determine the stage of the stream at the bridge site for the design discharge.

3. Plot representative cross section of stream for design discharge at Section 1, if not already done under Step 2. If the stream channel is essentially straight and the cross section substantially uniform in the vicinity of the bridge, the natural cross section of the stream at the bridge site may be used for this purpose.

4. Subdivide the above cross section according to marked changes in depth of flow and roughness. Assign values of Manning's roughness coefficient, n, to each subsection. Careful judgment is necessary in selecting these values.

5. Compute conveyance and then discharge in each subsection.

6. Determine the value of the kinetic energy coefficient.

7. Plot the natural cross section under the proposed bridge based on normal water surface for design discharge and compute the gross water area (including area occupied by piers).

8. Compute the bridge opening ratio, M, observing modified procedure for skewed crossings.

9. Obtain the value of K_b from the appropriate base curve.

10. If piers are involved, compute the value of J and obtain the incremental coefficient, ΔK_p.

11. If eccentricity is severe, compute the value of eccentricity and obtain the incremental coefficient, ΔK_e (FHWA 1978).

12. If a skewed crossing is involved, observe proper procedure in previous steps, then obtain the incremental coefficient, ΔK_s, for proper abutment type.

13. Determine the total backwater coefficient, K^*, by adding incremental coefficients to the base curve coefficient, K_b.

14. Compute the backwater by Equation 29.

15. Determine the distance upstream to where the backwater effect is negligible.

Detailed steps illustrated by examples are presented in *Hydraulics of Bridge Waterways* (FHWA 1978).

11.3 Inadequate Openings

The engineer will often encounter existing bridges and culverts that have been designed for storms having return periods less than 100 years. In addition, bridges will be encountered which have been improperly designed. Often the use of the orifice formula will provide a quick determination of the adequacy or inadequacy of a bridge opening:

$$Q_m = C_b A_b \sqrt{2gH_{br}}$$

(Equation 33)

or

$$H_{br} = 0.04\left(\frac{Q_m}{A_b}\right)^2$$

(Equation 34)

in which:

Q_m = the major storm discharge (cfs)

C_b = the bridge opening coefficient (0.6 assumed in Equation 33)

A_b = the area of the bridge opening (ft^2)

g = acceleration of gravity (32.2 ft/sec^2)

H_{br} = the head, that is the vertical distance from the bridge opening center point to the upstream water surface about 10H upstream from the bridge, where H is the height of the bridge, in feet. It is approximately the difference between the upstream and downstream water surfaces where the lower end of the bridge is submerged.

These expressions are valid when the water surface is above the top of the bridge opening.

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